CanonieEnvironmental

September, 1987   86-059

Annotated Kyl

GENERAL SITE INFORMATION

U.S. EPA REQUEST
AUGUST 31, 1987
WAUKEGAN HARBOR PROJECT

CanonieEnvironmental

OMC

OUTBOARD MARINE CORPORATION

September 22, 1987

Rodger Field, Esq.
Associate Regional Counsel
Region 5, U.S.EPA
230 South Dearborn Street
16th Floor
Chicago, Illinois 60604

Re: **Waukegan Harbor - OMC**

Dear Rodger:

At the conclusion of our technical meeting on August 31, you and other EPA representatives requested that OMC provide support documentation concerning certain technical aspects of the IPC remedy discussed in that meeting. The five items are:

1. Infiltration calculations for the IPC system;

2. Background information supporting the use of a permeability value of 1×10^{-7} cm/sec for the clay till underlying the site;

3. Slurry wall performance characteristics and permeability values;

4. Background information concerning the piezometric water levels in the Silurian Aquifer, near the site;

5. Details of the collection/dewatering well system.

Enclosed is a General Site Information document prepared by our consultants, Canonie Environmental Services and Golder and Associates, which responds to each of your request items.

We appreciate the full and frank discussions that occurred at the August 31 meeting, and welcomed the opportunity to meet
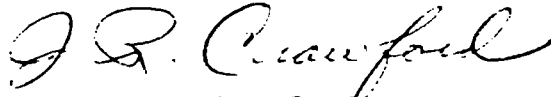
with your technical staff to address any outstanding questions associated with the December 1986 Waukegan Harbor Remedial Action Plan IPC proposal. We believe that each of the questions or comments raised by EPA at this meeting, as well as our earlier technical meetings, was either answered during the meeting, or has been addressed in our document submittals.

We look forward to further discussions in the near future toward resolving this matter.

If you or other EPA representatives have any questions regarding our response, please feel free to call me.

Sincerely,

J. Roger Crawford
Corporate Director
Environmental Control

JRC/je
Enclosures

cc: D. McArdle
     L. Baker

CanonieEnvironmental

September 21, 1987

86-059

Mr. Roger Crawford
Director of Environmental Control
Outboard Marine Corporation
100 Seahorse Drive
Waukegan, IL 60085

General Site Information
U.S. EPA Request
August 31, 1987
Waukegan Harbor Project

Dear Roger:

Canonie Environmental Services Corp. (Canonie) is enclosing the information requested by the U.S. Environmental Protection Agency (U.S. EPA) on August 31, 1987, for your review and submittal. The information includes:

1. Steady-State Flow Calculations for Closed In-Place Containments: Appendix A includes calculations by Golder and Associates, Inc. (Golder), for combined inflow from both the bottom and sides of the IPC containment cells. The total calculated inflow is 325 gallons per day when the assumed permeabilities of the slurry wall and clay till are 10^{-7} cm/sec and the head difference is 2 feet. The Golder calculations also assume that the head differential between the bedrock aquifer and the in-place containments is negligible. A second calculation by Canonie assumes that the piezometric level in the bedrock aquifer is greater than in the in-place containment. Based on nearby water supply wells, (see Item 4 herein) the estimated maximum excess piezometric head difference is approximately 20 feet. Under this assumption and with the other assumptions identical to the Golder calculation, the flow (470 gpd) remains less than 500 gallons per day.

2. Permeability of Clay Till: Appendix B includes information on actual laboratory and in-situ permeabilities of the Wadsworth till member of the Wedron formation, which underlies Lake County, Illinois. This information includes Pages 8 through 12 of Geology for Planning in Lake County, Illinois, 1973, describing the geologic classification of the clay till member located beneath the Outboard Marine Corporation (OMC) site. The information also includes laboratory permeability results from soil samples recovered from this formation approximately five miles from the OMC

site, between the city of Waukegan and the city of Gurnee, Illinois, and laboratory and in-situ permeability tests (a measure of horizontal permeability) taken at a landfill complex near Lake Calumet in the Wadsworth till member of the Wedron formation. These values indicate a laboratory vertical permeability of approximately 10^{-8} cm/sec and an average in-situ horizontal permeability of around 10^{-6} cm/sec.

T. A. Prickett, et al., 1964, reported that the vertical permeability of the confining bed separating the limestone bedrock aquifer and surficial sand and gravel deposits in the Libertyville area of northeastern Illinois has a permeability of 0.009 gpd/ft^2. This is equivalent to approximately 4×10^{-7} cm/sec.

Twelve published papers on ground water supply and geology in northwestern Illinois were reviewed by Golder and Canonie during past work on the OMC project. A list of these references is included in Appendix B. Pertinent papers and excerpts discussing the clay till permeability and the piezometric levels in the bedrock aquifer are included in Appendix B. These papers are:

    A. Hughes, G. M., et al., 1969;

    B. Larsen, J. I., 1973;

    C. Prickett, T. A., et al., 1964 (section on Libertyville);

    D. Woller, D. M., et al., 1976 (sections closest to Waukegan).

3.   Performance of Soil-Bentonite Slurry Walls: Appendix C includes information on the permeability and performance of soil-bentonite slurry walls. Included are Canonie's results of laboratory permeability tests and information on the actual in-situ performance of the slurry wall installed in San Jose, California, as discussed by Phil Antommaria at our meeting with the U.S. EPA on August 31, 1987.

The results indicate that the average laboratory permeability of the sand and gravel mixed with only 3 percent bentonite was 4 times 10^{-8} cm/sec. The San Jose slurry wall with an average depth of 106 feet and a total perimeter of 3,454 feet allows the maintenance of a constant drawdown within the slurry wall at a pumping rate of only 20 gallons per minute. Again, this wall was installed in a gravel and cobble formation having a permeability of 10^{-1} cm/sec.
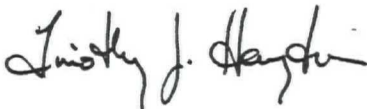
Appendix C also includes a curve of soil-bentonite permeability versus bentonite content for several basic soil types. The curve was developed by D'Appolonia, 1980.

CanonieEnvironmental

Normal construction quality control for a soil-bentonite slurry wall in a relatively clean uniform sand deposit, such as that found in the Waukegan Harbor area, will result in an in-place wall with a permeability of 10^{-7} to 10^{-8} cm/sec. The actual flow through the wall will depend on other critical factors such as head differential across the wall, actual wall thickness, and head loss in the filter cake zone next to the wall.

4. <u>Piezometric Level Limestone Aquifer</u>: Appendix D includes piezo-metric water levels in the limestone aquifer from the vicinity of a former city of Waukegan landfill north and west of the OMC site. The approximate well locations are shown on a topographic map and piezometric levels are interpreted based on depths from ground surface and the ground surface elevations shown on the USGS topographic map. The topographic map, well records, and a summary table are all presented in Appendix D.

5. <u>Proposed Dewatering Well Details</u>: Canonie is presently completing installation of a dewatering well system inside an in-place containment on a site in New York State. We have enclosed in Appendix E a typical plan of a dewatering well for this site. We believe that the OMC wells will be six- or eight-inch-diameter wells with wire-wound stainless steel screens in the bottom ten feet of the containment area. The design will be based on well spacing required to control the water level within the containment area and will contain at least three wells in each area. The exact locations, well details, and number of wells will be selected during final design.

Canonie believes this information should satisfy the requirements we understood from the August 31, 1987 meeting. If there are any questions, or additional information is required, please call.

Very truly yours,

Timothy J. Harrington
Director of Eastern Operations

TJH/tl

Attachments

CanonieEnvironmental

APPENDIX A

Golder Associates

CONSULTING GEOTECHNICAL AND MINING ENGINEERS

LETTER REPORT ON

TECHNICAL DOCUMENTATION

ONSITE CONTAINMENT

OUTBOARD MARINE CORPORATION

Submitted to:

Martin, Craig, Chester & Sonnenschein
55 West Monroe Street
Chicago, Illinois 60603

DISTRIBUTION:

1 Copy   - Martin, Craig, Chester & Sonnenschein
1 Copy   - Canonie
2 Copies - Golder Associates

September 1987   863-3389

Golder Associates

SUBJECT	OMC - ON SITE CONTAINMENT
Job No. 863-3389	Made by KP AKINS — Date 9/6/87
Ref.	Checked MRF 9-16-87 — Sheet 1 of 6
	Reviewed RSW

OBJECTIVE: DETERMINE QUANTITY OF FLOW INTO CONTAINMENT CELL WHILE MAINTAING FLOW INTO CELL — SLIP N° 3

METHOD: GENERAL REFERENCE: HARR, GROUNDWATER AND SEEPAGE, (1962) CHAPTERS 5 & 6.
SPECIFIC CITATIONS OF METHOD FOLLOW

ASSUMPTIONS:
1. GENERAL ASSUMPTION IS THAT POTENTIOMETRIC LEVEL IN THE SILURIAN AQUIFER IS AT OR ABOVE THE SURFICIAL (WATER TABLE) LEVELS

2. FOR ANALYSIS PURPOSES, TWO ASSUMPTIONS CAN BE CONSIDERED:

→ (1) TILL ZONE SUFFICIENTLY THICK THAT SILURIAN AQUIFER DOES NOT AFFECT SOLUTION,
— OR —
(2) SILURIAN AQUIFER ACTS AS IMPERMEABLE BOUNDARY (REPRESENTS HYDROSTATIC ASSUMPTION)

3. GEOMETRY OF CONDITIONS IS AS FOLLOWS:

SLIP NO. 3

75-300 ft

5'

30 ft SANDS   25'

S

TILL

$k = 1 \times 10^{-7}$ cm/sec

$$\left(\frac{1 f^2}{30.64 cm^2}\right)\left(\frac{8.64 \times 10^4 sec}{1 day}\right) \Rightarrow 2.8 \times 10^{-4} ft/d$$

Golder Associates

SUBJECT	OMC ON SITE CONTAINMENT		
Job No. 863-3389	Made by KP AKINS		Date 9/8/87
Ref.	Checked MRF 9-16-87		Sheet 2 of 16
	Reviewed R&W		

ANALYSIS METHOD I / ASSUMPTION (1) — THICK TILL LAYER

$$[\text{SEE HARR, pp 132 ff}]$$

$$d = 0 \qquad s = 5 ft \text{ and } 10 ft$$

$$2b = 75 \text{ AND } 300 \text{ FT} \quad (\text{SLIP 3})$$

$$I_E = 1.0 \text{ and } 0.5 \quad (\text{ASSUMED})$$

ALTERNATE ASSUMPTION IS 5-FT 'DIFFERENTIAL' HEAD
OUTSIDE TO INSIDE

TOTAL FLOW, Q

$$(\text{LENGTH}) \times (\text{UNIT FLOW}, q_r)$$

FOR END EFFECT, ADD ½ OF END LENGTH TO CELL LENGTH

— SEE NEXT PAGES FOR TABULATED SOLUTIONS —
COMBINATIONS OF h and s CONSIDERED
— CITED REFERENCE FOLLOWS TABLES

— OVAL LAGOON AND PARKING LOT ESTIMATES
INCLUDED

Golder Associates

SUBJECT	OMC PCB WAUKEGAN
Job No. 863-3369	Made by *mrt*
Ref.	Checked *KBA*
	Reviewed
Date	9/17/87
Sheet	3 of 6

TABULATED SOLUTION

VARIABLES		CONDITION/LOCATION			
		$h=2'$ SLIP NO.3	$h=2'$ SLIP NO.3		
s	EMBEDMENT (FT)	5	5		
$2b$	WIDTH (FT)	300	75		
s/b	RATIO (FT/FT)	0.033	0.133		
m	PARAMETER – FIG 5-33 [HARR]	0.94	0.76		
q/kh	FIG 5-35 [HARR]	3.2	2.2		
I_E	EXIT GRADIENT (FT/FT)	—	—		
$(I_E)(s)/h$	RATIO – FIG 5-36 [HARR]	0.32	0.35		
h	$h = \dfrac{(I_E)(s)}{(I_E)(s)/h}$ (FT)	2	2		
q	UNIT FLOW FT^3/DAY/FT $\times 10^{-4}$	17.9	12.3		
Q	FLOW, FT^3/DAY	1.1	.74		
Q	FLOW, GALLONS/DAY $Q = (LENGTH)(q)$	8.2	5.5		

$L \approx 600'$

AVG ≈ 7 gal/day

Golder Associates

SUBJECT	OMC PCB WAUKEGAN		
Job No. 863-3389	Made by *mr*		Date 9/17/87
Ref.	Checked KPA		Sheet 4 of 6
	Reviewed		

$k = 2.0^{40}$

TABULATED SOLUTION

VARIABLES		CONDITION / LOCATION			
		$h=2'$ PARKING LOT(2)	$h=2'$ PARKING LOT(1)	$h=5'$ PARKING LOT(2)	$h=5'$ PARKING LOT(1)
s	EMBEDMENT (FT)	5	5	5	5
$2b$	WIDTH (FT)	340	370	340	370
s/b	RATIO (FT/FT)	.029	.027	.029	.027
m	PARAMETER - FIG 5-33 [HARR]	0.97	0.98	0.97	0.98
$\frac{q}{kh}$	FIG 5-35 [HARR]	4.1	4.3	4.1	4.3
I_E	EXIT GRADIENT (FT/FT)	—	—	—	—
$(I_E)(s)/h$	RATIO - FIG 5-36 [HARR]	0.32	0.31	0.32	0.31
h	$h = \dfrac{(I_E)(s)}{(I_E)(s)/h}$ (FT)	2	2	5	5
q	$\left(\dfrac{q}{kh}\right)(k)(h)$ UNIT FLOW $\begin{Bmatrix} FT^3/DAY/FT \\ \times 10^{-4} \end{Bmatrix}$	23.0	24.1	57.4	60.3
Q	FLOW, FT^3/DAY	1.1 (2)	1.5 (1)	2.6	3.6
Q	FLOW, GALLONS/DAY $Q = LENGTH(q)$	8.2	11.2	19.4	26.9
Q_T	TOTAL Q FOR CONTAINMENT	19.4		46.3	

APROX END EFFECT

(1) $Q = l_1 q_1 + l_2 q_1 = (420)(2.41) + (185)(2.41) = 1.4$ FT^3/DAY

(2) $Q = l_3 q_2 + l_4 q_2 = (460)(23.9) + 0 = 1.1$ FT^3/DAY

Golder Associates

SUBJECT	OMC PCB WAUKEGAN
Job No. 863-3369	Made by _mrl_ Date 9/17/67
Ref.	Checked KPA Sheet 5 of 6
	Reviewed

TABULATED SOLUTION

VARIABLES		CONDITION / LOCATION			
		$h=2'$ ① OVAL LAGOON	$h=2'$ ② OVAL LAGOON	$h=5'$ ① OVAL LAGOON	$h=5'$ ② OVAL LAGOON
s	EMBEDMENT (FT)	5	5	5	5
$2b$	WIDTH (FT)	240	350	240	350
s/b	RATIO (FT/FT)	.042	.029	.042	.029
m	PARAMETER – FIG 5-33 [HARR]	0.95	0.98	0.95	0.98
q/kh	FIG 5-35 [HARR]	3.3	4.3	3.3	4.3
I_E	EXIT GRADIENT (FT/FT)	—	—	—	—
$(I_E)(s)/h$	RATIO – FIG 5-36 [HARR]	0.32	0.31	0.32	0.31
h	$h = \dfrac{(I_E)(s)}{(I_E)(s)/h}$ (FT)	2	2	5	5
q	UNIT FLOW $FT^3/DAY/FT$ $\times 10^{-4}$	18.5 / 18.4 (1)	24.1 (2)	46.2	60.2
Q	FLOW, FT^3/DAY	0.64	1.14	1.62	2.86
Q	FLOW, GALLONS/DAY $Q = LENGTH(q)$	4.8	8.5^6	12.1	21.4
Q_T	Q TOTAL FOR CONTAINMENT	13.5^4		33.5	

(1) $Q = q_1 l_1 + q_1 l_2 = (18.5)(230) + (18.4)(120) = 0.64$

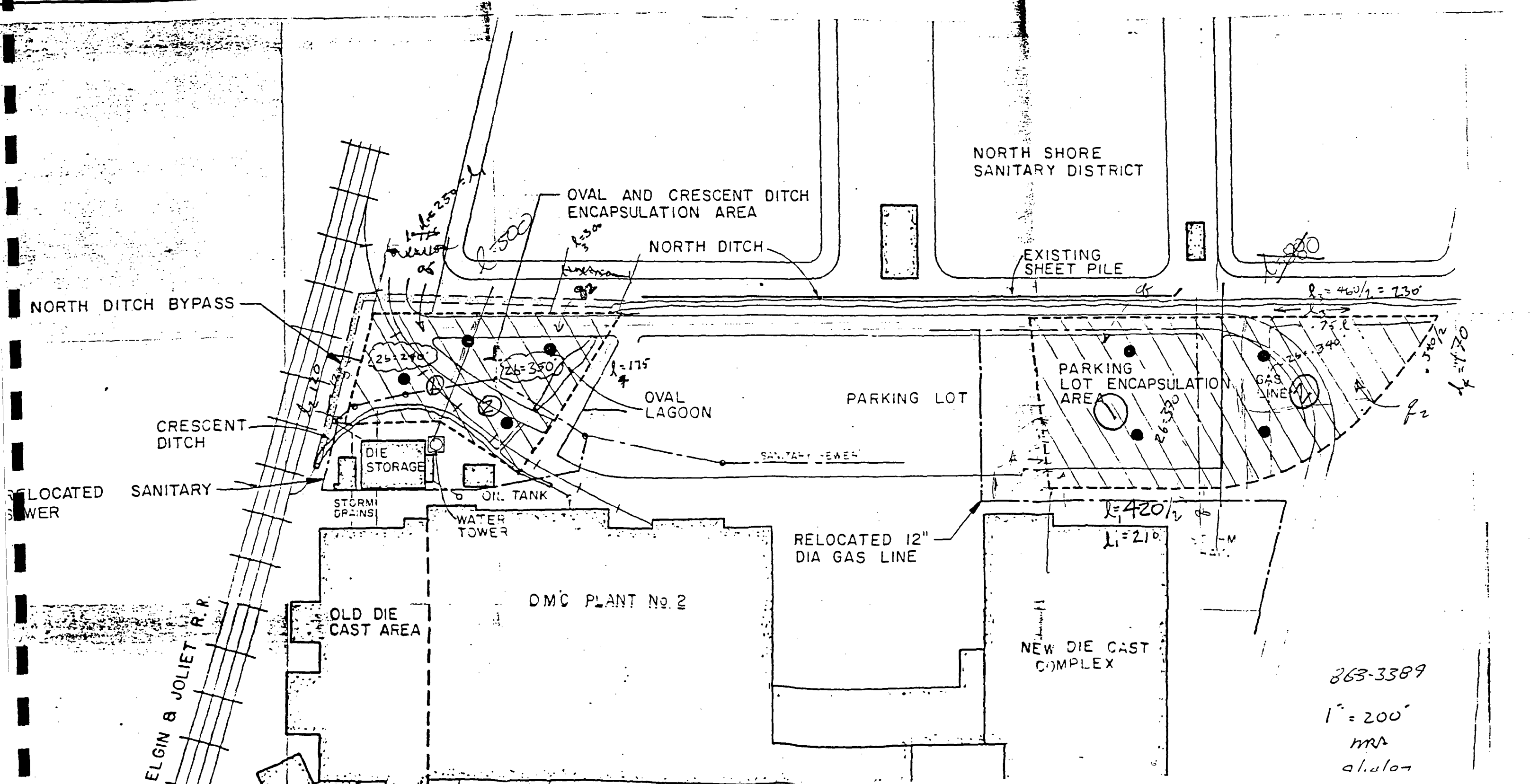(2) $Q = q_1 l_1 + q_1 l_4 = (24.1)(200) + (24.1)(175) = 1.14$

(ORIGINAL DRAWING NUMBER 84-036-E3)

CH 86-059-E10

REVISIONS	NO.		GWB	CHECKED BY				DRAWING
	DATE		10-30-86	APPROVED BY				NUMBER CH86-

NORTH SHORE
SANITARY DISTRICT

OVAL AND CRESCENT DITCH
ENCAPSULATION AREA

NORTH DITCH

EXISTING
SHEET PILE

NORTH DITCH BYPASS

$l_3 = 460/2 = 230'$

PARKING
LOT ENCAPSULATION
AREA

GAS
LINE

$l = 175$

OVAL
LAGOON

PARKING LOT

CRESCENT
DITCH

SANITARY SEWER

RELOCATED   SANITARY

DIE
STORAGE

OIL TANK

STORM
DRAINS

WATER
TOWER

RELOCATED 12"
DIA GAS LINE

$l = 420/2$

$l_1 = 210$

OMC PLANT No. 2

OLD DIE
CAST AREA

NEW DIE CAST
COMPLEX

ELGIN & JOLIET R.R.

863-3389

$1" = 200'$

mrs

a/.a/07

Golder Associates

SUBJECT	OMC / PCB / WAUKEGAN		
Job No. 863-3389	Made by KPA		Date 9/17/87
Ref.	Checked KPA		Sheet 1 of 2
	Reviewed		

HORIZONTAL FLOW INTO CONTAINMENTS

OBJECTIVE ESTIMATE QUANTITY OF FLOW HORIZONTALLY INTO CONTAINMENTS ASSUMING A 2-FOOT HEAD LOSS AND A ==2.5 FOOT THICK WALL==

METHOD FOR CONCEPTUAL ESTIMATE, ASSUME DARCY ✓ FLOW:

$$Q = k i A$$

FLOW = (PERMEABILITY)(GRADIENT)(AREA)

ASSUMPTIONS

WALL THICKNESS 2.0 FEET
HEAD LOSS 2 FEET
PERMEABILITY 1×10^{-7} cm/sec $= 2.8 \times 10^{-4}$ ft/day
 NOTE: ==PERMEABILITY EXPECTED TO BE 1×10^{-8} TO 1×10^{-10} cm/sec. CONSERVATIVE VALUE CHOSEN==

DIMENSIONS OF CELLS FROM DECEMBER 1, 1986 DOCUMENT, CANONIE DWG. CH 86-059-E34

WATER LEVEL 25 FEET ABOVE TILL (NOMINAL)

CALCULATIONS

3 ft wall.
2/3 =

SLIP 3 $Q = k i A$

$i = \dfrac{head\ loss}{wall\ thickness} = \boxed{2/2 = 1}$ ✓

$A = 2(600 \times 25) + 25(75) + 25(300) = 39,375\ sf$ ✓

$Q = 1 \times 10^{-7}\ cm/s \times (1) \times (39,375)$
$Q = \boxed{11.0\ ft^3/day}$ ✓ $\Rightarrow$ $7\frac{1}{3}\ ft^3/day$

$Q = \underline{82.3\ gal/day}$ $= 54.86\ gal/day$

OVAL LAGOON

$A = 25 \times Perimeter = 25 \times 1,600 = 40,000\ sf$

$Q = (2.8 \times 10^{-4}\ ft/day)(1)(40,000\ sf)$
$Q = 11.342\ cf/day = 7.47\ ft^3/d$
$Q = \underline{83.7\ gal/day} = 55.86\ gal/d$

Golder Associates

SUBJECT	OMC/PCB/Waukegan	
Job No. 863-3389	Made by mrl	Date 9/17/87
Ref.	Checked KM	Sheet 2 of 2
	Reviewed	

PARKING LOT CONTAINMENT AREA

$A = 25 \times 2,270 \, ft = 56,750 \, SF$ ✓

$Q = (2.8 \times 10^{-4} \, ft/day)(1)(56,750 \, sf)$

$Q = 15.89 \, cf/day$ $1589 = 10.59 \, ft^3/d$

$Q = \underline{119 \quad gal/day}$ $= 79.25 \, gal/d$

Golder Associates

SUBJECT	OMC / PCB / WAUKEGAN	
Job No. 863-3389	Made by MRA	Date 5/14/87
Ref.	Checked	Sheet 6 of 7
	Reviewed	

COMBINED FLOWS

CONTAINMENT	HORIZONTAL FLOW THROUGH WALL	UNDERSEEPAGE	TOTAL
SLIP No 3 ($n=2, s=5$)	82.4 gal/day	7 gal/day	89.4 gal/c
OVAL LAGOON ($n=2, s=5$)	83.7 gal/day	13.4 gal/day	97.1 gal/c
PARKING LOT CONTAINMENT AREA ($n=2, s=5$)	119 gal/day	19.4 gal/day	138.4 ga
		TOTAL	324.9 gpd

for a 2 ft wall
$i = 1$; for
a 3 ft thick
wall

$i = 2/3$

$Q = k i A$

$\therefore Q_{3} = Q_{2FT} (2/3)$

$\approx 216.60 \, gpd$

upstream side and below by the same amount on the downstream portion. A somewhat similar relationship is observed between the exact solution and the approximation (dotted line) for the structure assumed to be resting on the surface (considering pressures due to elevation head separately). Studies conducted by Pavlovsky with various ratios of $d/2b$ ($d/2b = 0$, $\frac{1}{10}$, $\frac{1}{5}$, $\frac{1}{3}$, 1.0, 3.0, 5.0) indicate that the difference between the pressures computed from the exact theory and the above approximation for $x \leq \pm 3b/4$ will be less than 12 per cent for $d/2b = \frac{1}{5}$. For $d/2b = \frac{1}{3}$, the difference will be less than 15 per cent.

5-8. Depressed Structure on Permeable Base of Infinite Extent with Two Symmetrical Rows of Pilings

The correspondence between the z plane and the t plane is shown in Fig. 5-32. Hence

$$z = M \int_0^t \frac{(t^2 - \sigma^2)\, dt}{\sqrt{(1 - t^2)(m^2 - t^2)}} \qquad (1)$$

Writing the numerator of the integrand as $(t^2 - m^2) + (m^2 - \sigma^2)$, we can split the integral into

$$z = -M \int_0^t \sqrt{\frac{m^2 - t^2}{1 - t^2}}\, dt + \frac{M(m^2 - \sigma^2)}{m} \int_0^t \frac{dt}{\sqrt{(1 - t^2)(1 - t^2/m^2)}}$$

Recognizing the first integral to be the same as Eq. (1), Sec. 5-7, and the
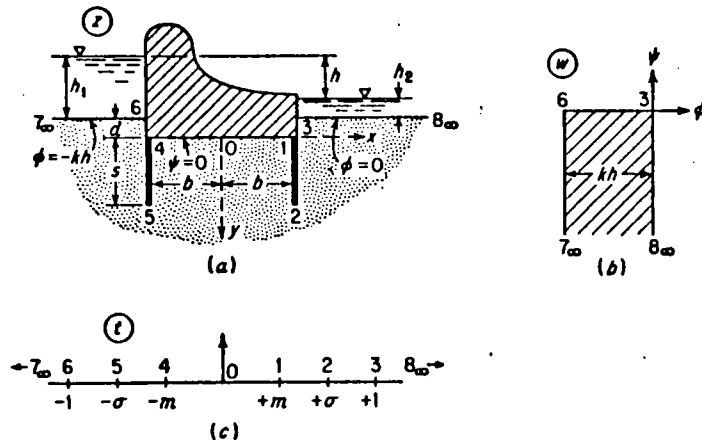
Fig. 5-32

second integral as the elliptic integral of the first kind of modulus $1/m$, we obtain

$$z = -M[(\sigma^2 - 1)F(m,\theta) + E(m,\theta)] \qquad (2)$$

where $\theta = \sin^{-1}(t/m)$. We note that, using Jacobi's notation, Eq. (2) can also be written as

$$z = -M[(\sigma^2 - 1)u + E(u)] \qquad (3)$$

where sn $u = t/m$.

To determine the unknown constants M, σ, and m, we consider the relationship between the points 1, 2, and 3 in the z and t planes.

At points 1, $z = b$ and $t = m$; hence sn $u = t/m = 1$, $u = K$, $E(u) = E(K) = \mathbf{E}$, and

$$b = -M[(\sigma^2 - 1)K + \mathbf{E}] \qquad (4)$$

At points 2, $z = b + is$ and $t = \sigma$; hence sn $u = \sigma/m > 1$ but $< 1/m$, which demonstrates that u for this case will be a complex number of the form $u = K + iv$ [cf. Eqs. (12), Sec. B-2]. Thus

$$\frac{\sigma}{m} = \text{sn}\,(K + iv) = \frac{1}{\text{dn}}\,(v,m')$$

and $v = \text{dn}^{-1}(m/\sigma,m')$ [Eq. (12a), Appendix B]. From Eq. (19b), Appendix B we have

$$E(u) = E(K + iv) = \mathbf{E} + i[v - E(m',v) + m'^2\,\text{sn}\,v'\,\text{cn}\,v'/\text{dn}\,v']^*$$

hence for points 2 we obtain

$$b + is = -M[(\sigma^2 - 1)K + \mathbf{E}]$$
$$\qquad\qquad - Mi\left[\sigma^2 v + \frac{m'^2\,\text{sn}\,v'\,\text{cn}\,v'}{\text{dn}\,v'} - E(m',v)\right] \qquad (5)$$

At points 3, $z = b - id$ and $t = 1$; hence sn $u = 1/m$, $u = K + iK'$, $E(u) = E(K + iK') = \mathbf{E} + i(K' - \mathbf{E}')$, and

$$b - id = -M[(\sigma^2 - 1)K + \mathbf{E}] - Mi(\sigma^2 K' - \mathbf{E}') \qquad (6)$$

On the basis of the above, we find that the three constants M, m, and σ can be determined from the equations

$$M[(\sigma^2 - 1)K + \mathbf{E}] = -b \qquad (7a)$$

$$M\left[\sigma^2 v + \frac{m'^2\,\text{sn}\,v'\,\text{cn}\,v'}{\text{dn}\,v'} - E(m',v)\right] = -s \qquad (7b)$$

$$M(\sigma^2 K' - \mathbf{E}') = d \qquad (7c)$$

* sn v', cn v', and dn v' designate sn (v,m'), cn (v,m'), and dn (v,m').

Combining Eqs. (7a) and (7c), we have for σ^2,

$$\sigma^2 = \frac{E' + (d/b)(K - E)}{K' + (d/b)K} \tag{8}$$

and hence σ can be determined once m is known. Substituting $(\sigma^2 - 1)$ from Eq. (8) into Eq. (7a), we obtain

$$-\frac{M}{b} = \frac{K' + (d/b)K}{E'K + EK' - KK'}$$

which, recognizing Legendre's formula, $E'K + EK' - KK' = \pi/2$, yields

$$M = -\frac{2b}{\pi}\left(K' + \frac{d}{b}K\right) \tag{9}$$

Substituting Eq. (7a) for M into Eq. (7b), we find

$$\frac{s}{b} = \frac{\sigma^2 v + (m'^2 \, \text{sn} \, v' \, \text{cn} \, v'/\text{dn} \, v') - E(m',v)}{(\sigma^2 - 1)K + E} \tag{10}$$

The R.H.S. of this expression was shown to be a function of the modulus and the ratio of d/b only. Hence this expression can be plotted to yield the modulus as a function of the ratios s/b and d/b. Such a plot was obtained by Harr and Deen [53] and is given in Fig. 5-33. We note in Eq. (8) that when $s = 0$, $m = \sigma$ and

$$\frac{d}{b} = \frac{E' - m^2 K'}{E - m'^2 K} \tag{11}$$

which is precisely the expression we obtained in Sec. 5-7 [Eq. (4)] for the depressed structure without pilings.

Finally, substituting for M and $(\sigma^2 - 1)$ into Eq. (3), we obtain for the required transformation between the z and t planes,

$$z = \frac{2b}{\pi}\left[\left(E' - K' - \frac{d}{b}E\right)u + \left(K' + \frac{d}{b}K\right)E(u)\right] \tag{12}$$

where the modulus m is given in Fig. 5-33.

For the mapping of the w plane (Fig. 5-32) onto the t plane, we have again [Eq. (6), Sec. 5-7],

$$t = \cos\frac{\pi w}{kh}$$

Hence the pressure in the water is

$$p = \gamma_w\left(\frac{h}{\pi}\cos^{-1}t + y + h_2 + d\right) \qquad 0 \leq \cos^{-1}t \leq \pi \tag{13}$$

(a)

(b)

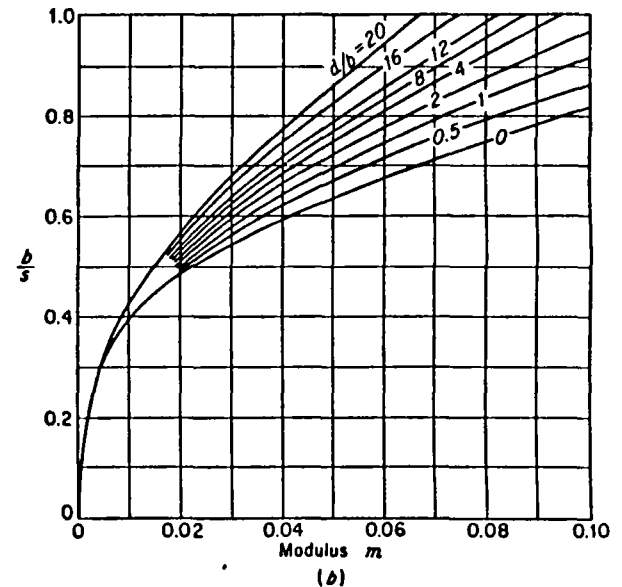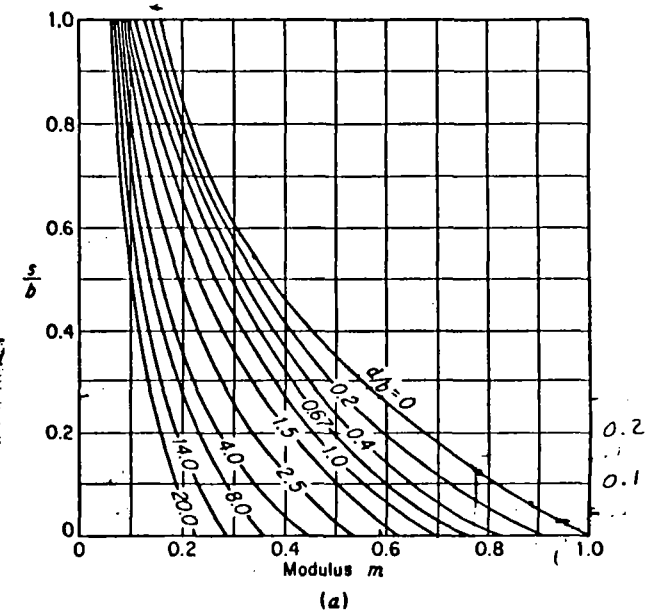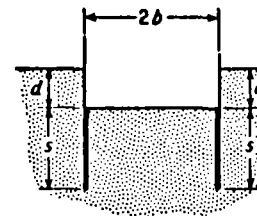Fig. 5-33

Once again it is advisable to use an indirect approach (assuming t and finding the corresponding z) for the determination of the pressure distribution along the contour of the structure (see Example 5-2).

For the exit gradient (point 3), using Eq. (2), Sec. 5-1, we readily obtain

$$I_s = \frac{hm'}{2b\left(K' - E' + \dfrac{d}{b}E\right)} \tag{14}$$

where the modulus is as is given in Fig. 5-33.

5-9. Double-wall Sheetpile Cofferdam

Figure 5-34 represents a section through a double-wall cofferdam consisting of two rows of sheetpiles. After the sheetpiles are driven, the
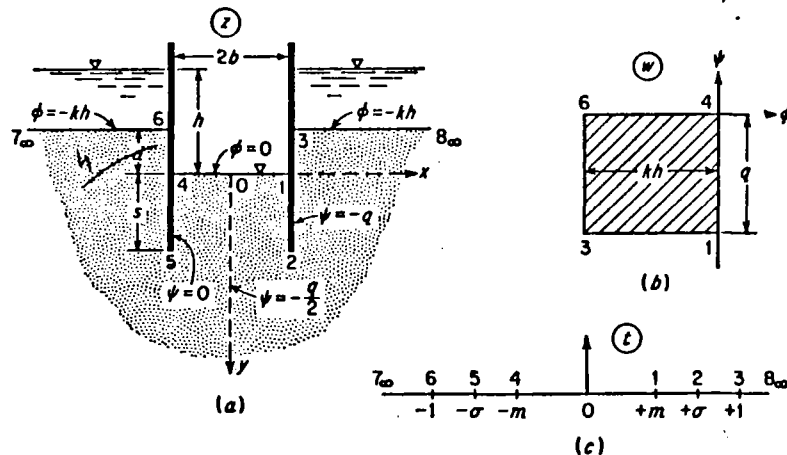
FIG. 5-34

soil between them is excavated to a depth d below the ground surface. We seek in this problem to determine the discharge quantity and the factor of safety with respect to piping.

Noting in Fig. 5-34 that the z plane and t plane are precisely the same as in Sec. 5-8, we have immediately for the required transformation between them [Eq. (12), Sec. 5-8],

$$z = \frac{2b}{\pi}\left[\left(E' - K' + \frac{d}{b}E\right)u + \left(K' + \frac{d}{b}K\right)E(u)\right] \tag{1}$$

where sn $u = t/m$, and the modulus m can be obtained directly from Fig. 5-33.

It is convenient in this problem to take the w plane as shown in Fig. 5-34b. Hence, for the mapping of the w plane onto the t plane, we have

$$w = \frac{M}{m}\int_0^t \frac{dt}{\sqrt{(1 - t^2)(1 - t^2/m^2)}} - \frac{iq}{2} = Mu - \frac{iq}{2} \tag{2}$$

where, as above, sn $u = t/m$.

Considering the correspondence at points 1, $t = m$ and $w = -iq$; hence sn $u = 1$, $u = K$ and

$$M = -\frac{iq}{2K} \tag{3}$$

At points 3, $t = 1$ and $w = -kh - iq$; hence sn $u = 1/m$, $u = K + iK'$, and

$$q = \frac{2khK}{K'} \tag{4}$$

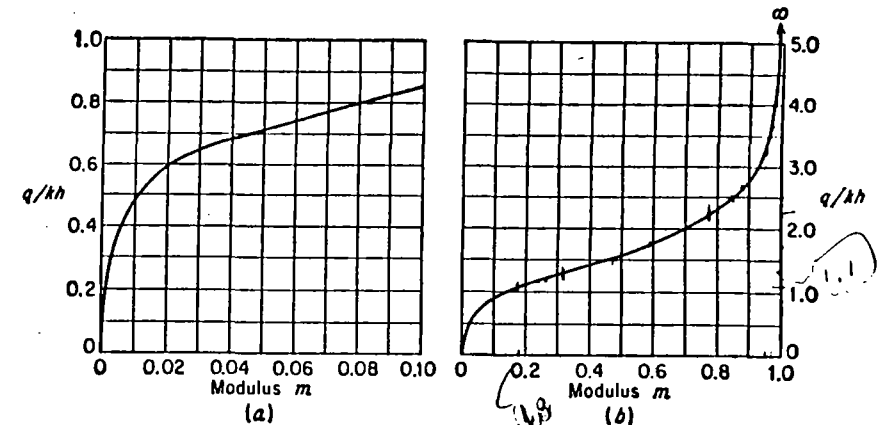A plot of Eq. (4) as a function of the modulus (Fig. 5-33) is given in Fig. 5-35.

FIG. 5-35

Recalling that $q = kIA$, where A is the area of the section normal to the direction of flow, we find, for the average exit gradient along the bottom of the excavation,

$$I_{av} = \frac{q}{2kb} \tag{5}$$

For the determination of the maximum exit gradient along the base of the excavation (at points 1 and 4, $t = \pm m$), from Eq. (2), Sec. 5-1, we obtain the relation

$$I_s = I_1 = I_4 = \frac{h\pi}{2bK'[K' + (d/b)K](m^2 - c^2)} \tag{6}$$

where σ^2 is defined by Eq. (8), Sec. 5-8. A plot of Eq. (6) in terms of $I_E s/h$ is given in Fig. 5-36.
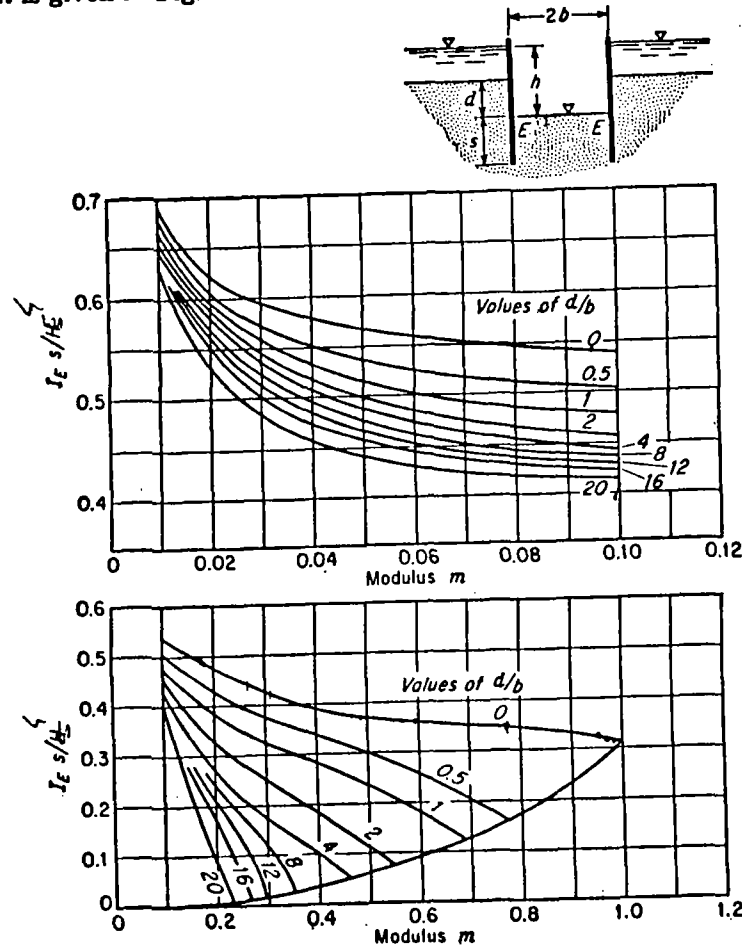

FIG. 5-36

Example 5-3. In Fig. 5-34, $h = 10$ ft, $d = 4$ ft, $2b = 40$ ft, and $s = 10$ ft. Determine (a) the reduced quantity of flow (q/k), (b) the average exit gradient, and (c) the maximum exit gradient.

From Fig. 5-33, with $s/b = 0.5$ and $d/b = 0.2$, we obtain the modulus $m = 0.35$. Then, from Fig. 5-35, we find $q/kh = 1.3$ and hence $q/k = 13$ ft.

From Eq. (5), we obtain the average gradient $I_{av} = {}^{13}\!/_{40} = 0.32$.

Next, entering Fig. 5-36 with $m = 0.35$ and $d/b = 0.2$, we find $I_E s/h = 0.39$, whence $I_E = 0.39$. Thus the factor of safety with respect to piping will be $1/0.39 \approx 2.6$.

PROBLEMS

1. Show that the transformation Eq. (3), Sec. 5-2, is valid for the points A and G of Fig. 5-2a.

2. Obtain the pressure distribution along all impervious boundaries in Fig. 5-4.

3. Obtain the general expression for the uplift force for a weir resting on the ground surface (of infinite depth) with a centrally placed sheetpile.

4. Demonstrate that each of the exit-gradient formulas in Fig. 5-9 can be obtained from Eq. (16), Sec. 5-2.

5. Verify that the complete mapping of the z plane onto the t plane in Fig. 5-10 is given by Eq. (1), Sec. 5-3.

6. Show that with $\gamma = 90°$, Eq. 5, Sec. 5-3, yields the transformation for a vertical sheetpile.

7. Derive the general expression for the exit gradient for an inclined sheetpile and discuss the nature of this gradient when γ in Fig. 5-11 is (a) equal to $\frac{1}{2}$, (b) less than $\frac{1}{2}$, and (c) greater than $\frac{1}{2}$.

8. Noting that the rectangle (w plane) in Fig. 5-13b becomes a semi-infinite strip as points A and D approach infinity, demonstrate that Eq. (5), Sec. 5-4, will degenerate into $t = \cos(\pi w/kh)$.

9. A 20 ft wide weir (without piling) rests on the surface of a 15-ft layer of soil. The head loss is 15 ft. Obtain the distribution of the factor of safety with respect to piping along the tail-water boundary for a distance of 50 ft downstream of the toe of the structure.

10. Verify Eqs. (14), Sec. 5-5.

11. Verify Eqs. (15), Sec. 5-5.

12. For the section shown in Fig. 5-37, obtain the general expression for (a) the quantity of seepage, (b) the exit gradient, and (c) the pressure distribution along the piling.
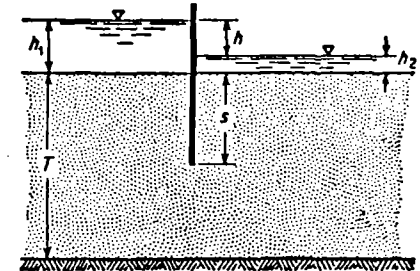

FIG. 5-37

13. Solve Prob. 3 for a layer of finite thickness T, and compare with the solution to Prob. 3.

14. For the section shown in Fig. 5-38, estimate the factor of safety with respect to (a) uplift force, (b) uplift moment (neglect moment due to piling), and (c) piping ($\gamma_w = 124.8$ pcf).
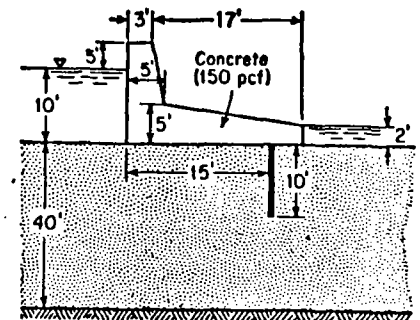

FIG. 5-38

For the exit gradient [Eq. (5b)], from Fig. 5-22 with $s/T = \frac{1}{3}$, we find $I_{Es}/h = 0.31$, where h as given in this figure is half the head loss in the type II fragment. Hence

$$I_E = \frac{0.31 \times 2 \times 5.93}{9} = 0.41$$

From Eq. (14), Sec. 5-8 (for an infinite depth of porous media), the exit gradient is found to be 0.43.

6-7. Flow in Layered Systems

Closed-form solutions for the seepage characteristics of even simple structures founded in layered media offer considerable mathematical difficulty. On the basis of her closed-form solutions for the sections shown in Fig. 6-21, in 1941 Polubarinova-Kochina [114] developed an
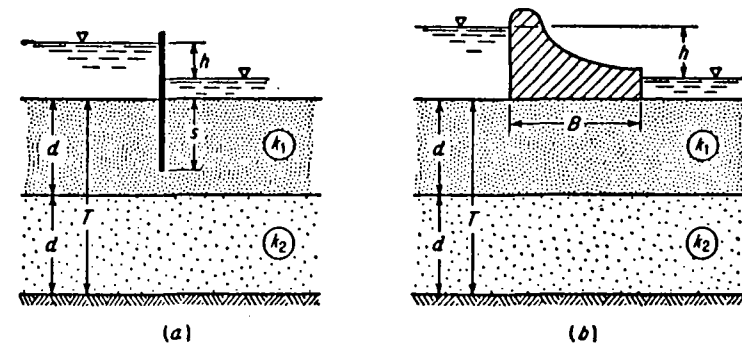

Fig. 6-21

approximate procedure whereby the seepage characteristics of structures founded in layered systems can be obtained simply and with a great degree of reliability. The procedure will be illustrated for the sections shown in Fig. 6-21 for which the exact solutions are known. In these figures the flow media consist of two horizontal layers of equal thickness d underlain by an impervious base. The coefficient of permeability of the upper layer is k_1, and of the lower layer k_2, and the coefficients of permeability are related to the dimensionless parameter ϵ by the expression

$$\tan \pi\epsilon = \sqrt{\frac{k_2}{k_1}} \qquad (1)*$$

Thus, as the ratio of the permeabilities varies from 0 to ∞, ϵ ranges between 0 and $\frac{1}{2}$.

Let us investigate the discharge and the exit gradient for the structures shown in Fig. 6-21 for some special values of ϵ.

* Compare with Sec. 1-14, Subsection 3.

Single Sheetpile Embedded in Two Layers of Equal Thickness and of Different Coefficients of Permeability (Fig. 6-21a)

1. $\varepsilon = 0$. When $\varepsilon = 0$, from Eq. (1) we have $k_2 = 0$, which is equivalent to having the impervious base at depth d. Hence for this case the flow region is reduced to a single homogeneous layer for which the discharge and exit gradient are known. From Eq. (14), Sec. 5-5, taking $b = 0$, we get for the discharge

$$q = \frac{k_1 h K'}{2K} \qquad m = \sin \frac{\pi s}{2d} \qquad (2)$$

From Eqs. (16), Sec. 5-5, the exit gradient is

$$I_E = \frac{h\pi}{4K\,dm} \qquad (3)$$

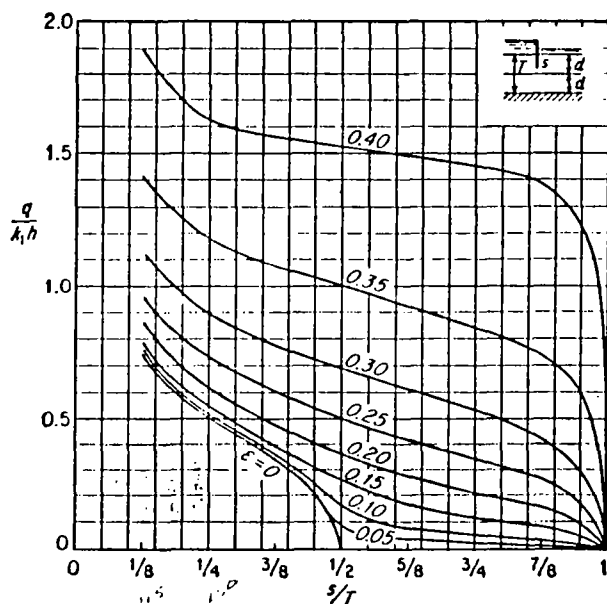In the above, $s < d$. If $s \geqq d$, obviously $q = I_E = 0$.

Fıg. 6-22. (*After Polubarinova-Kochina* [116].)

The curve for $q/k_1 h$ as a function of s/T for $\varepsilon = 0$, where T is the thickness of both layers, is given in Fig. 6-22. A crossplot is shown for various s/T ratios (at $\varepsilon = 0$) in Fig. 6-23. Similar plots for $I_E T/h$ are presented in Fig. 6-24.

2. $\varepsilon = \frac{1}{4}$. When $\varepsilon = \frac{1}{4}$, $k_2 = k_1$, and the system is reduced to a single homogeneous layer of thickness $2d$, for which Eqs. (2) and (3) are applicable (taking d in these expressions as $2d$). The corresponding plots for $\varepsilon = \frac{1}{4}$ are given in Figs. 6-22 to 6-24.
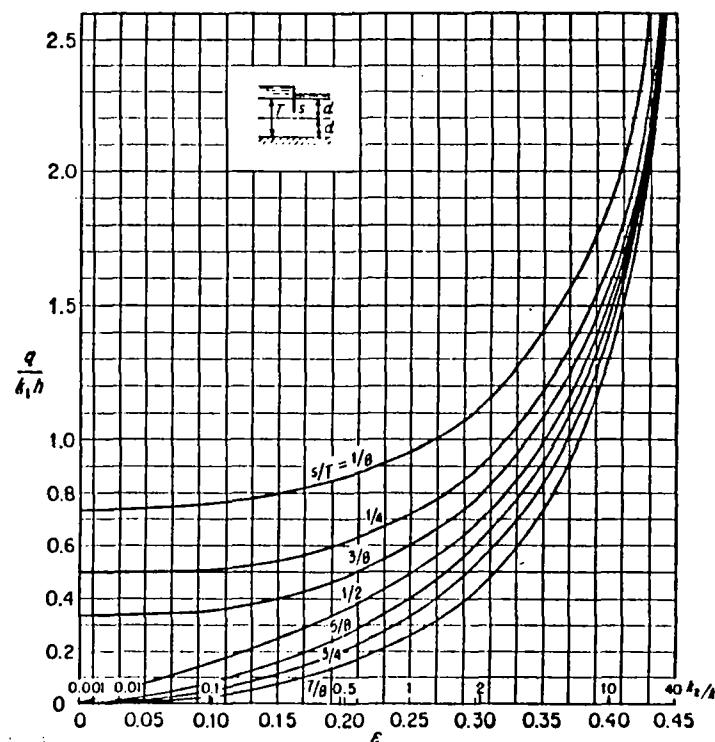

Fıg. 6-23. (*After Polubarinova-Kochina* [116].)

3. $\varepsilon = \frac{1}{2}$. When $\varepsilon = \frac{1}{2}$, $k_2 = \infty$, and there is no resistance to flow in the bottom layer. Hence the discharge through the section is infinite; i.e., $q = k_1 h K'/2K = \infty$. Now, as K'/K must be infinite for this case, for values of ε in the vicinity of $\frac{1}{2}$, Polubarinova-Kochina recommends the approximate equality of $K'/K = \tan \varepsilon\pi$. Thus, for values of ε close to $\frac{1}{2}$, the discharge can be determined by

$$q = \frac{k_1 h}{2} \sqrt{\frac{k_2}{k_1}} \qquad (4)$$

The closed-form solution yields for this case

$$\lim_{\varepsilon \to \frac{1}{2}} \left(\frac{q/k_1 h}{\tan \varepsilon\pi} \right) = \frac{1}{2}$$
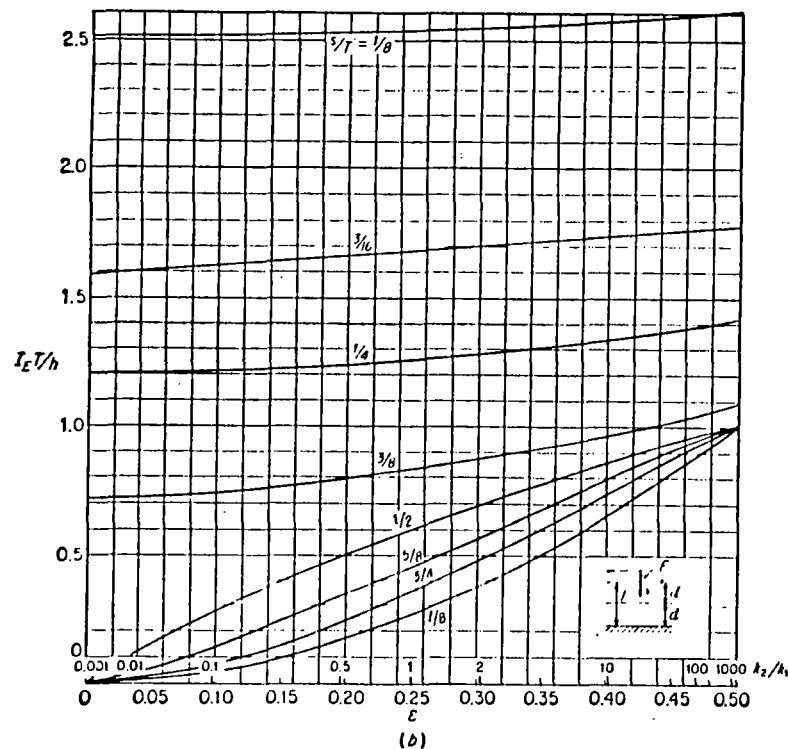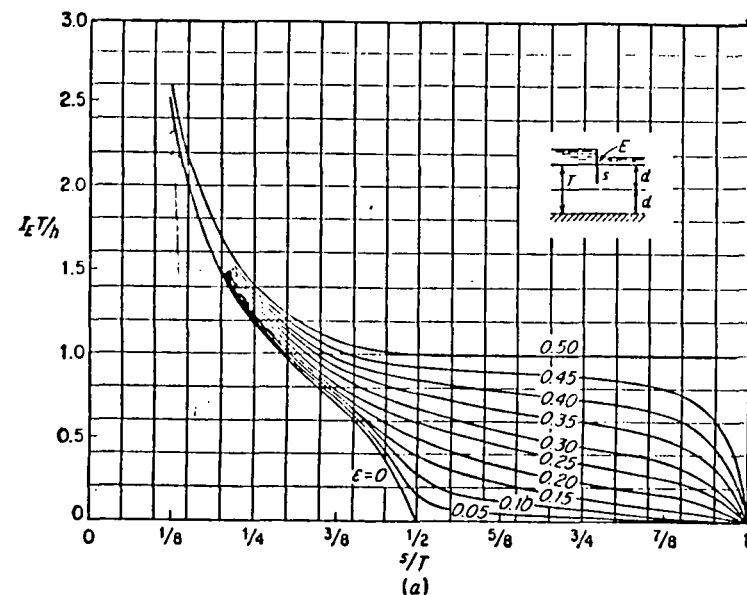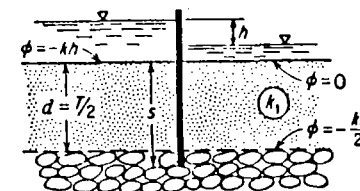
Fig. 6-24. (*After Polubarinova-Kochina* [116].)

Fig. 6-25

If $s \geq d$, the flow through the upper layer is one-dimensional (Fig. 6-25), and hence the exit gradient is $I_E = h/T$, or

$$I_E \frac{T}{h} = 1 \tag{5}$$

To determine the exit gradient for $s < d$, it is necessary to proceed as in Chap. 5 and find the correspondence between the z plane and the w plane in Fig. 6-26. The mechanics of the solution will be left for the

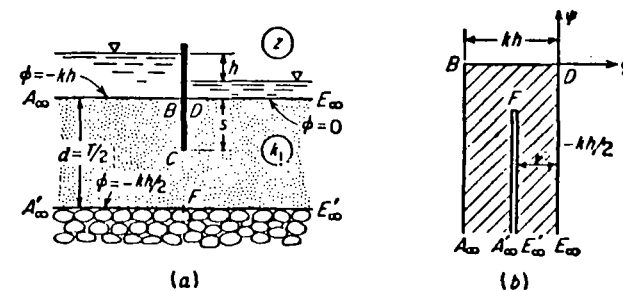

Fig. 6-26

problems.* The complex velocity for this problem is found to be ($T = 2d$)

$$u - iv = -\frac{k_1 h}{T} \frac{\cosh (\pi z/T)}{\sqrt{\sinh^2 (\pi z/T) + \sin^2 (\pi s/T)}} \tag{6}$$

and hence ($z = 0$ at the exit point)

$$I_E \frac{T}{h} = \frac{1}{\sin (\pi s/T)} \tag{7}$$

The results given in Eqs. (5) and (7) are shown in Fig. 6-24.

Flat-bottom Structure Resting on the Surface of Two Layers of Equal Thickness and of Different Coefficients of Permeability

For this section (Fig. 6-21b) the exit gradient will be unbounded for all cases; hence we need only be concerned with the question of the discharge as a function of the permeability ratios. For both $\varepsilon = 0$ and $\varepsilon = \frac{1}{4}$ we

* See Girinsky [43].

find that the discharge [Eqs. (14), Sec. 5-5, with $s = 0$] is given by

$$q = \frac{k_1 h K'}{2K} \qquad m = \tanh\frac{\pi B}{2d} \qquad (8)$$

where d is to be taken as $2d$ (or T) for $\epsilon = \frac{1}{4}$. Figure 6-27 shows the variation of this function for values of B/T where T is the thickness of both layers. For values of ϵ in the vicinity of $\frac{1}{2}$, Polubarinova-Kochina again recommends the use of the approximate relationship given in Eq. (4).

The foregoing demonstrates that for the special values of $\epsilon = 0$, $\epsilon_1 = \frac{1}{4}$, and $\epsilon = \frac{1}{2}$, the problem of determining the flow characteristics for structures founded in two layers of different permeabilities can be reduced to one for a single homogeneous layer. This provides the essentials of Polubarinova-Kochina's approximate method wherein exact solutions are obtained and plotted for values of $\epsilon = 0$ and $\epsilon = \frac{1}{4}$ and in the near vicinity of $\epsilon = \frac{1}{2}$, and smooth curves are drawn between these known points (such as in Figs. 6-23, 6-24b, and 6-27b), from which intermediate values can be obtained by interpolation.

A simplification in the above method when the discharge becomes infinite can be effected by plotting the inverse of the ordinate scale for the discharge; that is, plot the curves for $k_1 h/q$ rather than $q/k_1 h$ versus ϵ, as was done in Fig. 6-27b. This obviates the difficulty at $\epsilon = \frac{1}{2}$, which now becomes a known point ($k_1 h/q = 0$). For portions of curves such as $s/T = \frac{1}{2}$ in Fig. 6-23, where $q/k_1 h \to 0$ as $\epsilon \to 0$, part of the curves can be obtained from plots using $k_1 h/q$ (at $\epsilon = \frac{1}{4}$ and $\epsilon = \frac{1}{2}$), and part using $q/k_1 h$ (for $\epsilon = 0$ and $\epsilon = \frac{1}{4}$).

The above method can be used as well for layers of unequal thickness. For example, in Fig. 6-28 for $\epsilon = 0$, the equivalent depth of the flow domain is d, whereas at $\epsilon = \frac{1}{4}$ the depth is $3d$.
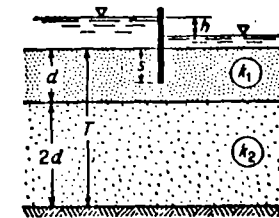

FIG. 6-28

Finally, it should be noted that this procedure can be combined with Pavlovsky's method of fragments to yield approximate solutions for even the most complicated structures.
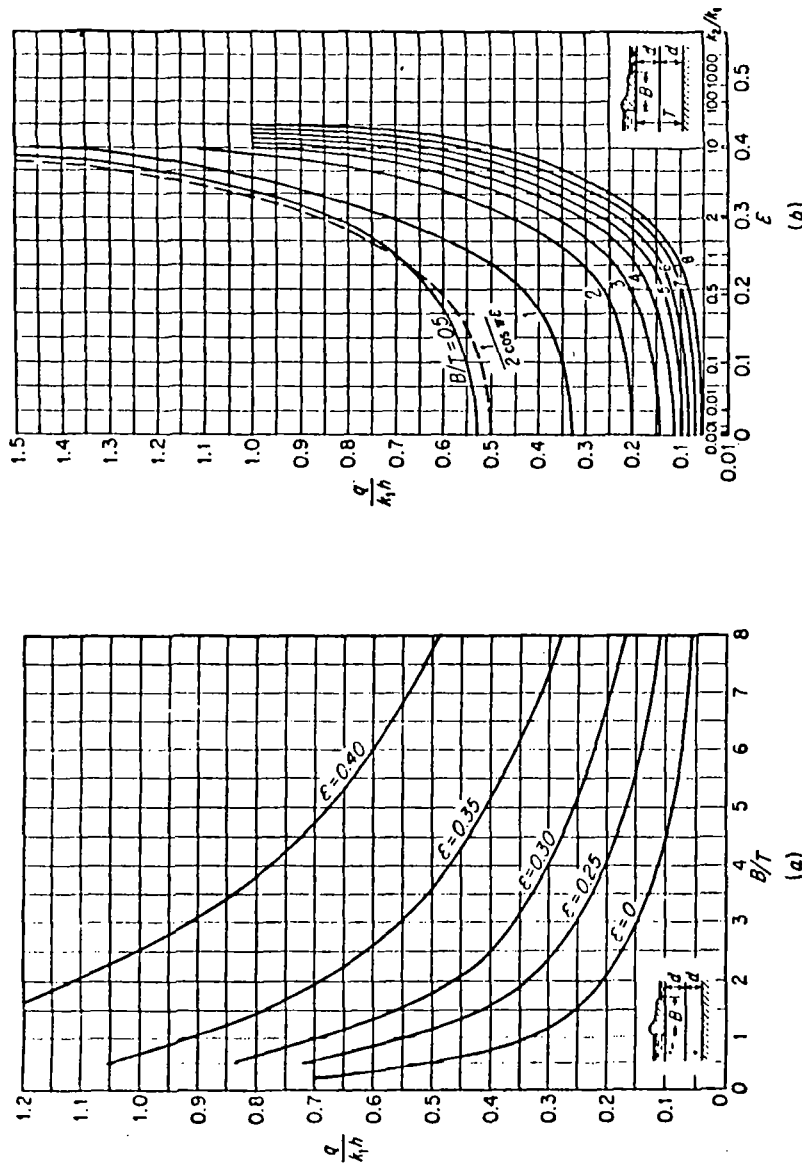

FIG. 6-27. (After Polubarinova-Kochina [116].)

Canonie

By _TJH_	Date _9-13-87_	Subject _Calculate Inflow to_	Sheet No. _1_ of _3_
Chkd. By _MJS_	Date _9-17-87_	_IPC; Steady State_	Proj. No. _86-059_

1/4" X 1/4"

.Purpose: Determine the approximate inflow to the three IPC cells proposed for Waukegan Harbor, after site capping and after steady state inflow is established, see figure page 3

Assumptions:

1. No inflow through the cap (HDPE liner)

2. Head difference between cell and underlying limestone aquifer is positive upwards

3. Both in-site slurry wall and underlying clay till are 10^{-7} cm/sec. permeability

4. Slurry walls are keyed 3 feet into till and are 2 ft wide

5. Head differentials are:

   Surficial sand   2 ft
   Limestone aquifer   20 ft

6. Containments are:

CONTAINMENT	DEPTH (FT) TO TILL	WETTED DEPTH (FT)	LENGTH (FT)	AREA (FT²)
SLIP # 3	22	19	1700	72,000
LAGOONS	25	22	1400	91,000
PARKING LOT	25	22	2100	234,000

Canonie

By __TJH__ Date _9-13-87_ Subject __Calculate Inflow to__ Sheet No. _2_ of _3_

Chkd. By _MJS_ Date _9-17-87_ __IPC, Steady State__ Proj. No. _86-059_

$$K = 2.8 * 10^{-4} \ ft/day \quad 1/4" \times 1/4"$$

FLOW THROUGH WALL $\quad Q = KiA \quad$ (NOTE $i = {}^{2FT}/_{2FT}$)

CONTAINMENT	WETTED AREA (FT²)	$Q \left(\frac{ft^3}{sec}\right)$	$Q \left(\frac{ft^3}{day}\right)$	$Q \left(\frac{gallons}{day}\right)$
SLIP # 3	32,300	1.6×10^{-4}	9.16	68
LAGOONS	30,800	1.0×10^{-4}	8.73	65
PARKING LOT	46,200	1.5×10^{-4}	13.05	97

$\frac{3 \ foot}{walls}$

$230 \ jpd = 153\frac{1}{3} \ gpd$

CHECK FOR UPFLOW FROM AQUIFER $\quad Q = KiA \quad$ (ASSUMING 1-D) $\quad i = {}^{20}/_{70} = 0.2$

CONTAINMENT	$Q \ \frac{ft^3}{sec}$	$Q \ \frac{ft^3}{day}$	$Q \ \frac{gallons}{day}$
SLIP # 3	6.7×10^{-5}	5.83	43
LAGOONS	8.5×10^{-5}	7.37	55
PARKING LOT	2.2×10^{-4}	18.9	141

239. gpd

BECAUSE OF UPWARD FLOW, THERE WILL BE NO FLOW FROM THE SHALLOW WATER TABLE UNDER THE WALL

∴ TOTAL Q = 230 + 239 ≈ __470 gpd__

Canonie

By **TJH** Date **9-13-87** Subject Calculate Inflow to — Sheet No. **3** of **3**

Chkd. By **MJS** Date **9-17-87** — IPC; Steady State — Proj. No. **86-059**

1/4" X 1/4"

CAP?

PIEZ. LEVEL 578'

PIEZ. LEVEL 580

SLURRY WALL

CLAY TILL

PIEZO. LEVEL 598'

BEDROCK AQUIFER

APPENDIX

B
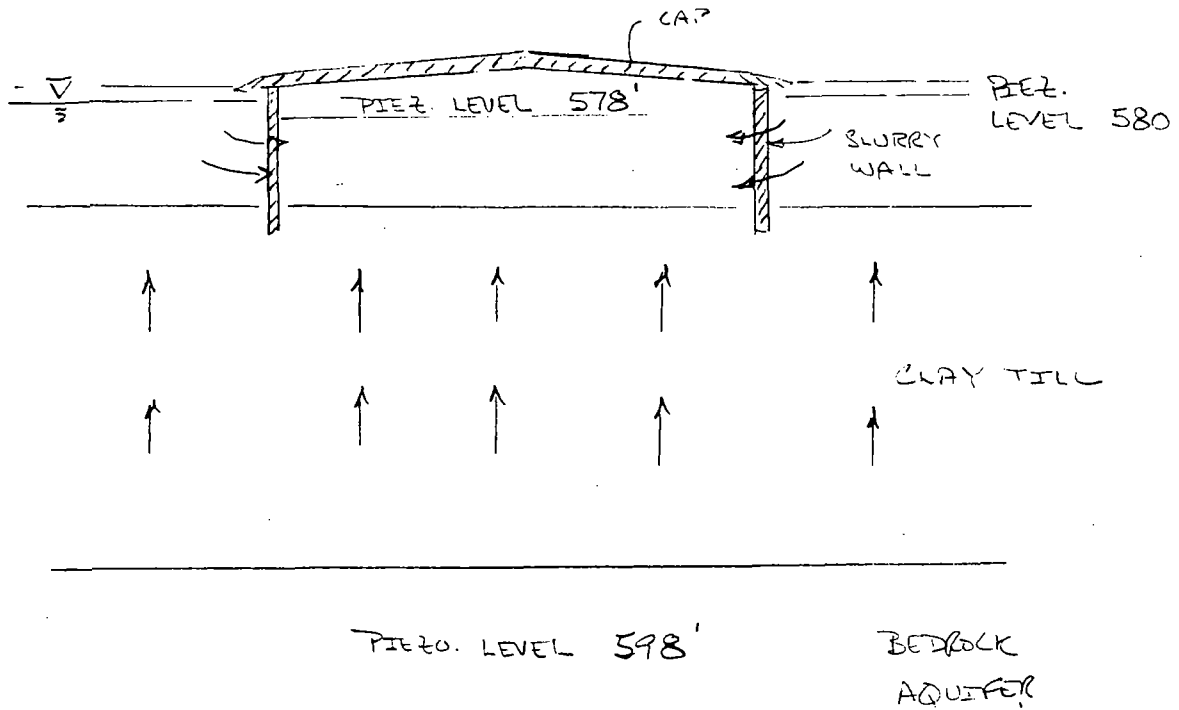
APPENDIX B

ACKNOWLEDGMENTS

Evaluation of the geologic units required the work of a number of Survey geologists. The writer mapped the surficial materials (pl. 1) and prepared the cross sections (pl. 2, A and B). Ross D. Brower, Jean Peterson Bogner, and Kemal Piskin prepared the ground-water maps (pl. 2, C and D); Norman C. Hester evaluated the sand and gravel resources (table 1 and pl. 3); Paul B. DuMontelle evaluated the conditions for general construction (table 1 and pl. 4); and George M. Hughes assisted the writer in the study of conditions for disposal of waste.

GEOLOGY

The landscape of Lake County has been shaped principally by the action of water and glacial ice. Layered, consolidated bedrock underlies the surficial unconsolidated deposits, which are generally between 100 and 300 feet thick (pl. 2, B - cross sections). The unconsolidated material is thickest— between 200 and 300 feet—in the western part of the county, whereas east of the Des Plaines River it is between 100 and 200 feet thick.

At present, the county is drained by the Fox River in the west and the Des Plaines River and its tributaries throughout the central area (pl. 1). The southeastern part of the county is drained by the North Branch of the Chicago River and its tributaries. A number of short, intermittent streams, many with very steep gradients, which flow into Lake Michigan, drain the eastern edge of the county.

The major landforms were shaped by the last glaciers to cover the county. In the western part of the county, these include gravelly hills (kames) about 75 feet high and 1000 feet in diameter interspersed with many lakes and bogs (pl. 1). East of the gravel hills and lakes is a broad, complex upland composed of pebbly, sandy, silty clay (glacial till) extending to the Des Plaines River Valley. This upland is part of the Valparaiso Morainic System (fig. 2), one of the most conspicuous topographic features in northeastern Illinois. This system is composed of a broad series of ridges (moraines), which is more than 10 miles wide. Undrained depressions are numerous among the ridges, and these contain either small lakes or small swamps filled with peat and muck.

The Des Plaines River Valley was a drainageway for glacial meltwaters, which deposited sand, silt, and gravel in the valley. East of the Des Plaines is another series of moraines, known as the Lake Border Morainic System. Along the Lake Michigan shore are sand and gravel deposits brought there by the wave and current action of the lake.

UNCONSOLIDATED SURFICIAL DEPOSITS

Unconsolidated deposits in Lake County range from about 75 feet to 300 feet in thickness. These deposits bear most of man's activities in Lake County. The majority of the unconsolidated deposits were left by the glaciers that formerly covered the region. The deposits are mapped and described on plate 1 and are discussed in this report. Emphasis is on their physical characteristics, which control the feasibility of the various land uses. Formal geographic names have
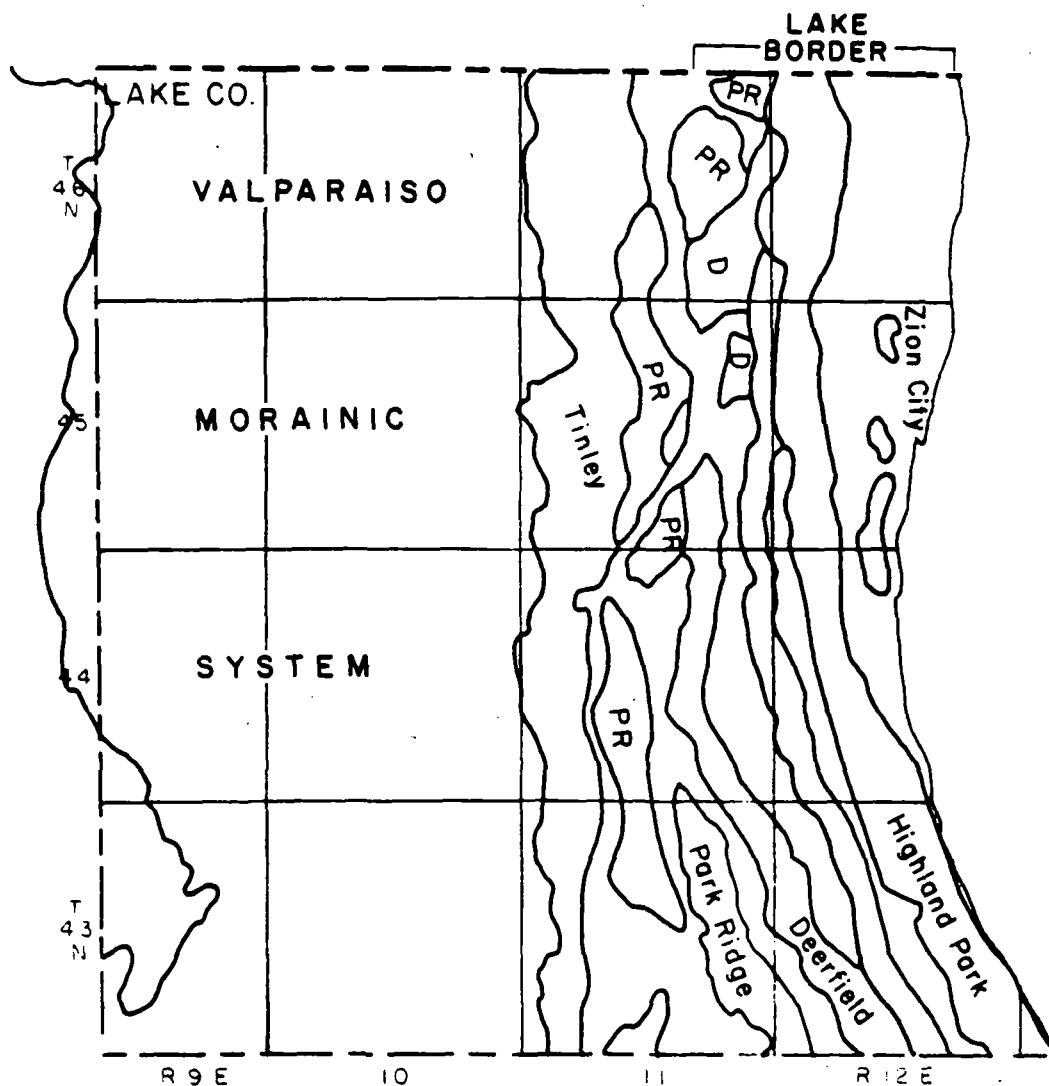
Fig. 2 - Moraines in Lake County (after Willman and Frye, 1970).

been designated for the deposits, following current stratigraphic practice, and the names conform to those used by the Illinois State Geological Survey (Willman and Frye, 1970). The names facilitate identification of the deposits in Lake County with equivalent deposits in other parts of Illinois.

Other geographic names that are used in connection with the deposits refer to prominent moraines that have been recognized in northeastern Illinois. For example, one of the more extensive deposits, the Wadsworth Till Member of the Wedron Formation (pl. 1) is subdivided geographically by referring portions of it to the Lake Border, Tinley, or Valparaiso Moraine (Willman, 1971, and fig. 2).

The deposits are described in the following pages in groupings of similar materials: glacial till, glacial sand and gravel, lake and bog deposits, wind-blown deposits, and river deposits. These are generally arranged by age from oldest to youngest. The deposits are also described in the legend of plate 1 with the oldest at the bottom and the youngest at the top. Figure 3 relates the named deposits to geologic time and also shows in cross section the physical relations of the deposits to each other. For convenience in locating certain kinds of earth materials, there is also a key in plate 1 to map units, which are arranged according to their texture (grain size) from finest to coarsest. Additional details on the mapped units mentioned in this report are given by Willman and Frye (1970) and Willman (1971).

Glacial Till

Wedron Formation

The largest proportion of the glacial deposits covering the bedrock in Lake County is glacial till. Till is unsorted, ice-deposited sediment composed of a matrix of silt, clay, and sand, in which pebbles, cobbles, and boulders are embedded. The cross sections on plate 2B show that there are four units that make up the bulk of the glacial deposits: a basal till unit with associated sand and gravel, which is not exposed at the surface in Lake County; an intermediate, locally rather extensive, sand and gravel deposit; an upper till, which is the principal surface material in about three-fourths of Lake County; and scattered, generally local deposits consisting principally of sand and gravel, water-laid silts, and peat. The two tills and the intermediate sand and gravel deposit are all included in the Wedron Formation, even though the precise identity of the basal till has not been determined.

Wadsworth Till Member (ws, wcs, wc, wsc)* — The principal surface unit, the Wadsworth Till Member, underlies all of Lake County except for small portions of the westernmost part of the county. It ranges in thickness from just a few feet to more than 250 feet. In general, it is characterized by a yellow or olive brown color in the upper 5- to 10-foot oxidized zone, which contains a surface-soil profile, and by gray below the zone of oxidation; it ranges in textural composition from clay to clayey silt or slightly sandy clayey silt; it is generally pebbly and contains a few boulders. The pebbles and boulders are most commonly dolomite and shale.

The textural variations noted within the Wadsworth Till are used to subdivide the till into the units mapped on plate 1 and in some cases roughly correspond to moraines (fig. 2). The area west of the Des Plaines River is underlain by till, the matrix of which is predominantly silt. The till has two mappable variations: a clayey phase (wcs) and a somewhat sandy phase (ws). The clayey phase occupies about half of Lake County, covering much of the central area from just west of the Des Plaines River to the sand and gravel area along the western border. This area corresponds generally to the tract mapped as the Valparaiso Morainic System. The topography of the surface of this till is morainic, with many low ridges and hills interspersed with lakes, lake basins, and peat bogs. The till is yellowish brown to gray-brown and contains numerous

---

* The letter symbols designate the various map units shown on plate 1.

TIME UNITS

SERIES	STAGE	SUBSTAGE
PLEISTOCENE		
	HOLOCENE	
	WISCONSINAN	VALDERAN
		TWOCREEKAN
		WOODFORDIAN

ROCK UNITS

Richland Loess

Cahokia Alluvium (c)

Grayslake Peat (g)

Lake Michigan Fm. Ravinia Mbr. (lr)

Wedron Fm. — Wadsworth Mbr. (wc, wsc, wcs, ws)

Henry Fm. — Mackinaw Mbr.(hm), Batavia Mbr.(hb), Wasco Mbr.(hw)

Equality Fm. — Carmi Mbr (ec), Dolton Mbr. (ed)

Cross-section labels:

Wedron Formation

Batavia Mbr. (hb)
Grayslake Peat (g)
Wasco Mbr (hw)
Mackinaw Mbr. (hm)
Cahokia Alluvium (c)
Dolton Mbr. (ed)
Carmi Mbr (ec)
Ravinia Mbr. (lr)
Lake Mich.

Fig. 3 - Geological classification and field relations of surficial materials in Lake County. Symbols for rock units are those used on plate 1.

carbonate pebbles and shale fragments. It is the thickest till unit in the county, averaging 50 to 60 feet. In some places, however, under the higher ridges, it may be as much as 150 feet thick (pl. 2). Within the same central area, there are patches of the sandy phase of the till (ws).

Most of the sandy phase (ws) of the silty till occurs in a relatively narrow band just west of the Des Plaines River and corresponds roughly to the Tinley Moraine, which forms a long narrow ridge averaging about a mile and a half in width. In this area the till may locally contain as much as 30 percent sand. It is brownish yellow, contains many dolomite pebbles, and averages 20 to 30 feet in thickness. The till overlies the clayey phase (wcs) to the west and appears to extend eastward in the subsurface beneath the more clayey tills (pls. 1 and 2). It crops out again at the surface along the lake shore at the base of the bluff along Lake Michigan. This band of outcrop is very narrow and could not be shown on plate 1, but the outcrop is shown on large-scale Soil Conservation Service maps (Paschke and Alexander, 1970).

The most clayey portions of the Wadsworth Till occur within the area mapped as the Lake Border Morainic System on the eastern side of the county. The Lake Border Morainic System consists of five long, narrow, closely spaced moraines trending north and south, paralleling the shoreline of Lake Michigan. The moraines are called, from west to east, the Park Ridge, Deerfield, Blodgett, Highland Park, and Zion City. The clayey Wadsworth Till can be divided into two phases, a silty clay phase (wsc) and a clayey phase (wc). The clayey phase is prevalent in the southeastern quarter of the county, where it may contain as much as 70 percent clay. There, lacustrine sediments are commonly associated with the till. This clayey till is between 30 and 40 feet thick and is olive yellow, olive brown, and gray. It contains many shale and dolomite pebbles.

Although there is a possibility of transition between the various textural units within the Wadsworth Till Member, the cross sections (pl. 2) suggest that there are bodies of sand and gravel that may separate these units, even though the sand and gravel zones are relatively thin and generally local and discontinuous.

Glacial Sand and Gravel

Henry Formation

During periods of glacial melting, vast quantities of loose material were washed away in the meltwaters. This material, called outwash, is size-sorted. Fast-moving waters carried and deposited sand and gravel whereas slow-moving waters deposited fine sand and silt. Three types of outwash deposits are recognized in Lake County: valley deposits, outwash plains, and kames. All outwash deposits at the surface in Lake County are assigned to one of the three members of the Henry Formation.

Valley deposits (Mackinaw Member, hm) — Meltwater channeled down the Des Plaines River deposited sand and gravel outwash assigned to the Mackinaw Member of the Henry Formation (hm). Sand and gravel deposits along the river average 25 to 30 feet in thickness, and gravel pits have been operated in these deposits for many years.

TABLE B-1

PERMEABILITY OF WADSWORTH TILL MEMBER
OF THE WEDRON FORMATION

Site Location: Approximately 5 miles west of OMC site*

Boring No.	Field Value	Lab Value	Coefficient of Permeability	Calculations**
MW-1		X	5.7×10^{-8} cm/sec	X
MW-2		X	8.6×10^{-9} cm/sec	X

Site Location: CID Facility, Calumet City, IL

Boring No.	Field Value	Lab Value	Coefficient of Permeability	Calculations**
G-203-P	X		2.6×10^{-8} cm/sec	
G-215-P	X		1.1×10^{-7} cm/sec	
G-216-P	X		4.7×10^{-8} cm/sec	
G-218-P	X		3.1×10^{-6} cm/sec	
G-203-P		X	2.2×10^{-8} cm/sec	X
G-215-P		X	1.8×10^{-8} cm/sec	X
G-216-P		X	2.0×10^{-8} cm/sec	X
G-203, S-7		X	1.3×10^{-8} cm/sec	
G-203, S-9		X	6.1×10^{-9} cm/sec	
G-208, S-3		X	1.9×10^{-8} cm/sec	
G-208, S-11		X	4.9×10^{-9} cm/sec	
G-210, S-5		X	1.3×10^{-8} cm/sec	
G-210, S-7		X	7.1×10^{-9} cm/sec	

*Client is confidential

**Calculations are attached for indicated borings

Canonie

By _KMB_ Date _11/14/36_ Subject.

Chkd. By _DMA_ Date _-4-96_

Sheet No. _1_ of _3_

Proj. No. _26-26_

1/4" X 1/4"

PERMEABILITY TESTS

THE LABORATORY PERMEABILITY VALUES ARE CALCULATED ON THE FOLLOWING PAGES. THE TEST RESULTS ARE PRESENTED ON THE FIGURES WHICH SHOW THE FLOW VOLUME WITH THE TIME (t). THE CROSS SECTIONAL AREA (A) AND HEIGHT OF THE SAMPLE (L) WERE MEASURED IN THE LABORATORY. THE TOTAL HEAD LOSS (h) ACROSS THE SAMPLE WAS 35psi OR 2462 c.m. FOR BOTH TESTS.

Canonie

SAMPLE MW-1

SAMPLE LENGTH = 7.5 cm.
SAMPLE DIAMETER = 7.3 cm
CROSS SECTIONAL AREA = 41.9 cm²

INITIAL TIME = 0.0 HRS.
FINAL TIME = 122.4 HRS.
ELAPSED TIME = 122.4 HRS

INFLUENT FLOW = 344.1 cm³
EFFLUENT FLOW = 345.3 cm³
AVERAGE FLOW = 345.2 cm³

FLOW RATE

$$Q = \frac{V}{\Delta T} = \frac{345.0 \text{ cm}^3}{122.4 \text{ HRS}} = 2.82 \text{ cm}^3/\text{HR}.$$

COEFFICIENT OF PERMEABILITY

$$k = \frac{QL}{Ah} = \frac{(2.8 \text{ cm}^3/\text{HR})(7.5 \text{ cm})}{(41.9 \text{ cm}^2)(2462 \text{ cm})(3600 \text{ sec/HR})}$$

$$k = 5.7 \times 10^{-8} \text{ cm/sec}$$

Canonie

By KMB Date 11/14/86 Subject

Chkd. By DMA Date 11-14-86

Sheet No. 3 of

Proj. No. 86-06

1/4" X 1/4"

SAMPLE MW-2

SAMPLE LENGTH = 8.4 cm.
SAMPLE DIAMETER = 7.3 cm.
CROSS SECTIONAL AREA = 41.9 cm²

INITIAL TIME = 88.6 HRS.
FINAL TIME = 146.4 HRS
ELAPSED TIME = 57.8 HRS

INFLUENT FLOW = 23.9 cm³
EFFLUENT FLOW = 20.4 cm³
AVERAGE FLOW = 22.2 cm³

FLOW RATE

$$Q = \frac{V}{\Delta T} = \frac{22.2 \text{ cm}^3}{57.8 \text{ HRS}} = .38 \text{ cm}^3/\text{HR}.$$

COEFFICIENT OF PERMEABILITY

$$K = \frac{QL}{Ah} = \frac{(.38 \text{ cm}^3/\text{HR})(8.4 \text{ cm})}{(41.9 \text{ cm}^2)(2462 \text{ cm})(3600 \text{ SEC}/\text{HR})}$$

$$= 8.6 \times 10^{-9} \text{ cm}/\text{SEC}$$

LABORATORY PERMEABILITY TEST RESULTS
TUBE MW-1

Legend
□ INFLUENT FLOW
△ EFFLUENT FLOW

TOTAL FLOW (ML)

ELAPSED TIME (HOURS)

CanonieEnvironmental
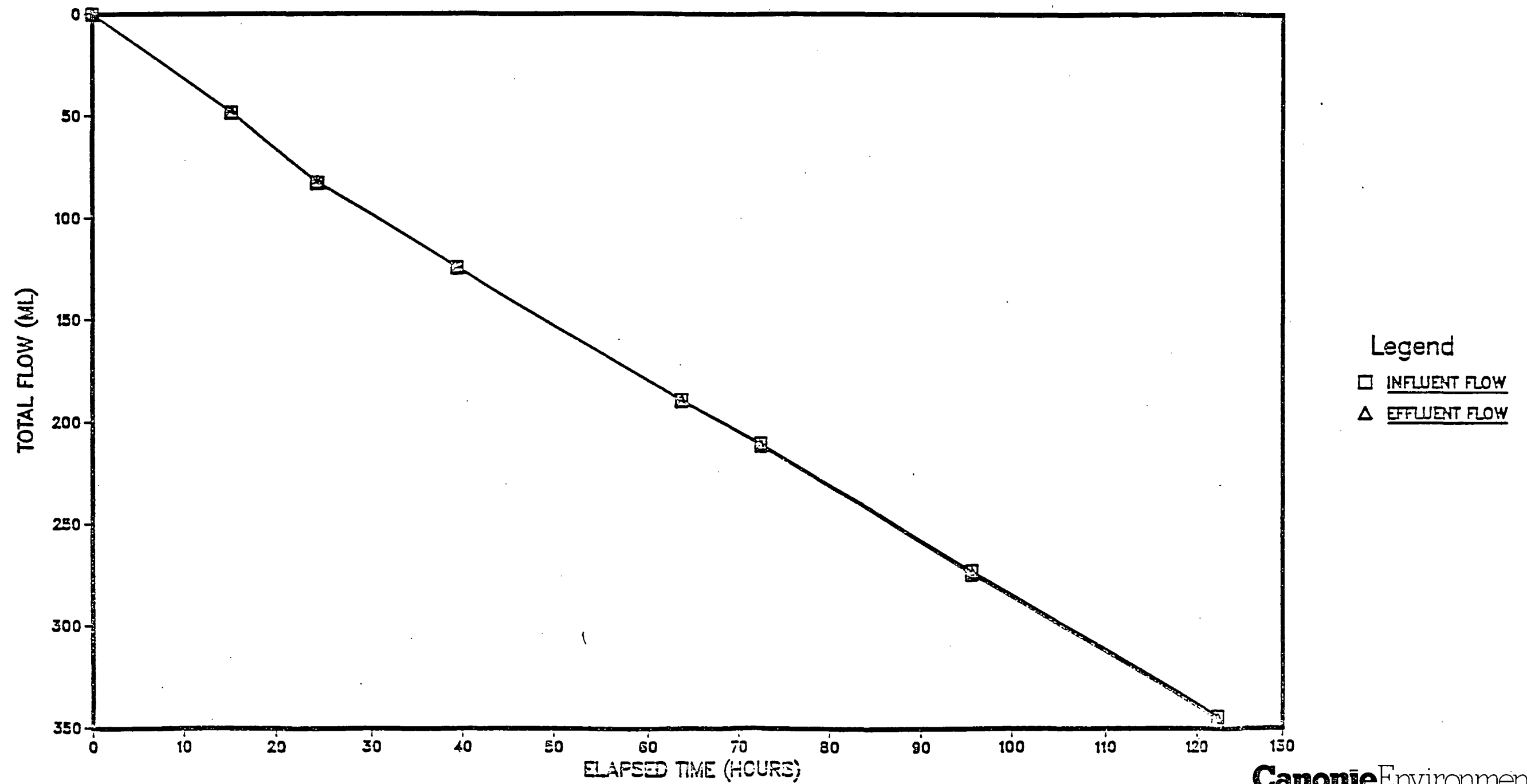
LABORATORY PERMEABILITY TEST RESULTS
TUBE MW-2, DEPTH OF 9.0 TO 12.0 FT.

Legend
□ INFLUENT FLOW
△ EFFLUENT FLOW

CanonieEnvironmental

Canonie
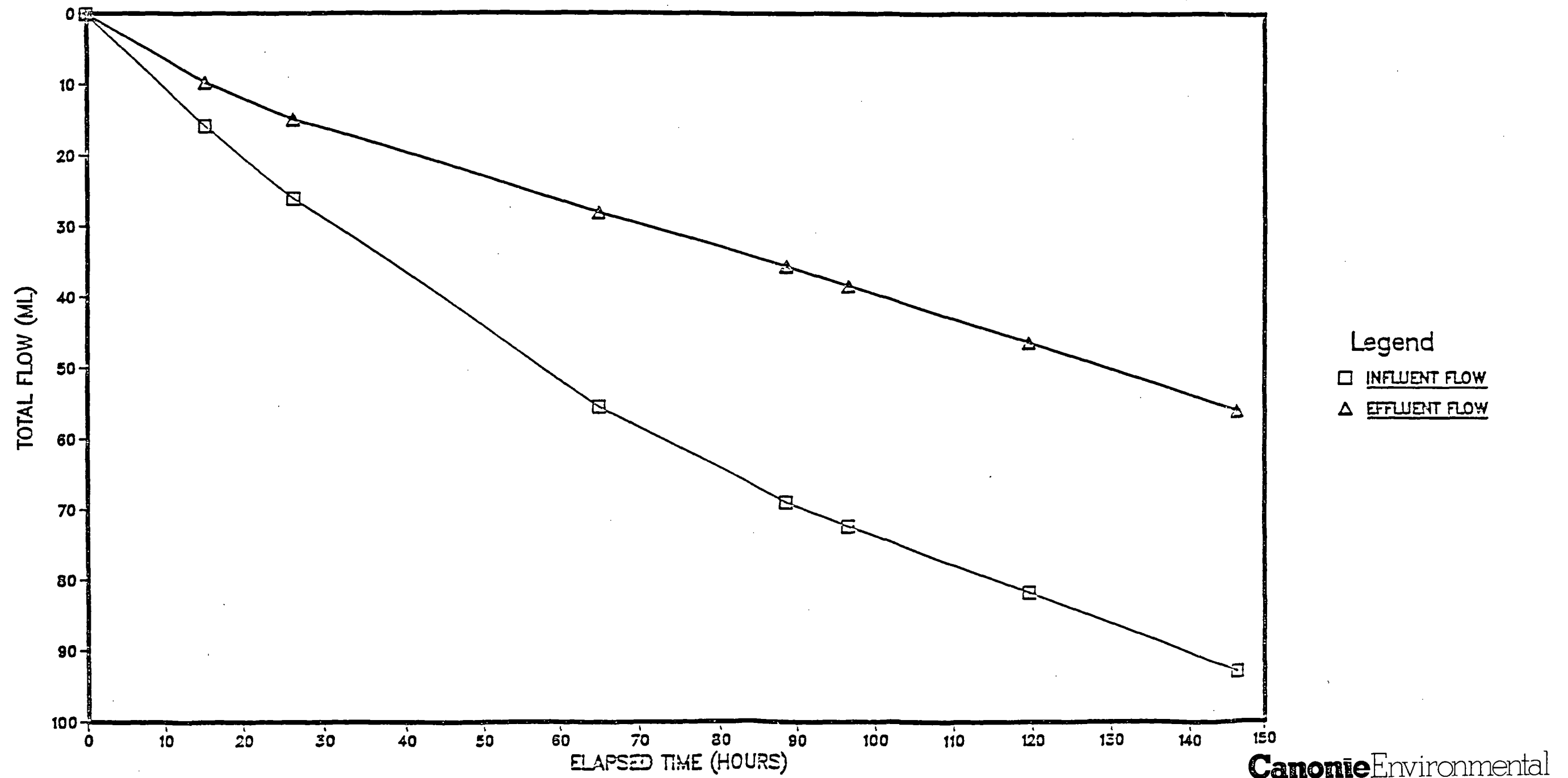
By **KMB** Date **3/23/87** Subject **WASTE MANAGEMENT** Sheet No. _____ of _____

Chkd. By _____ Date _____   **CID — AREA 4**   Proj. No. **86-089**

1/4" X 1/4"

PERMEABILITY TESTS

THE LABORATORY PERMEABILITY VALUES ARE CALCULATED ON THE FOLLOWING PAGES. THE TEST RESULTS ARE PRESENTED ON THE FIGURES WHICH SHOW THE FLOW VOLUME WITH THE TIME (t). THE CROSS SECTIONAL AREA (A) AND HEIGHT OF THE SAMPLE (L) WERE MEASURED IN THE LABORATORY. DISTILLED WATER WAS USED AS A PERMEANT FOR THE FIRST 150 HRS. OF THE TEST. AREA 4 LEACHATE WAS USED FOR THE REMAINDER OF THE TEST. THE TOTAL HEAD LOSS (H) ACROSS THE SAMPLE WAS 10 P.S.I. (703 cm.).

Canonie

By FAH   Date 3/23/87 Subject CID-AREA-4-203P-Sample 8   Sheet No. ___ of ___
Chkd. By DMA Date 3-23-87   PERMEABILITY COMPUTATIONS   Proj. No. CH86-??9

1/4" X 1/4"

De~~~ ~~~~ ~~~

SAMPLE LENGTH, L = 8.90 cm

SAMPLE DIAMETER = 7.20
CROSS-SECTIONAL AREA, A = 40.72 cm^2

INITIAL TIME, t_1 = 1303.27 hrs
FINAL TIME, t_2 = 2019.42 hrs
ELAPSED TIME, Δt = 716.15 hrs

VOLUME OF INFLUENT = 540.60 - 353.30 = 137.30 ml   (ml = cm^3)
VOLUME OF EFFLUENT = 515.00 - 333.70 = 181.30 ml
        AVERAGE = 184.30 cm^3

VOLUMETRIC FLOW RATE, $Q = \dfrac{\delta V}{\delta t} = \dfrac{184.30\ cm^3}{716.15\ hrs} \cdot \dfrac{1}{\frac{1\ hr}{3600\ sec}} = 7.15 \times 10^{-5}\ \frac{cm^3}{sec}$

PRESSURE DIFFERENTIAL, ΔP = 10 psi

DIFFERENTIAL HEAD, $H = \dfrac{\Delta P}{\gamma} = \dfrac{\left(10\ \frac{lb}{in^2}\right)\left(144\ \frac{in^2}{ft^2}\right)\left(30.48\ \frac{cm}{ft}\right)}{62.4\ \frac{lb}{ft^3}}$

$H = 703.4\ cm$

COEFFICIENT OF PERMEABILITY, $K = \dfrac{QL}{AH}$

$K = \dfrac{\left(7.15 \times 10^{-5}\ \frac{cm^3}{sec}\right) \cdot (8.90\ cm)}{(40.72\ cm^2) \cdot 703.4\ cm}$

$\underline{K = 2.2 \times 10^{-3}\ cm/sec}$

LABORATORY PERMEABILITY TEST RESULTS
WMI–CID AREA 4 – BORING 203P SAMPLE 8

Legend
□ INFLUENT FLOW
△ EFFLUENT FLOW

CanonieEnvironmental

Canonie
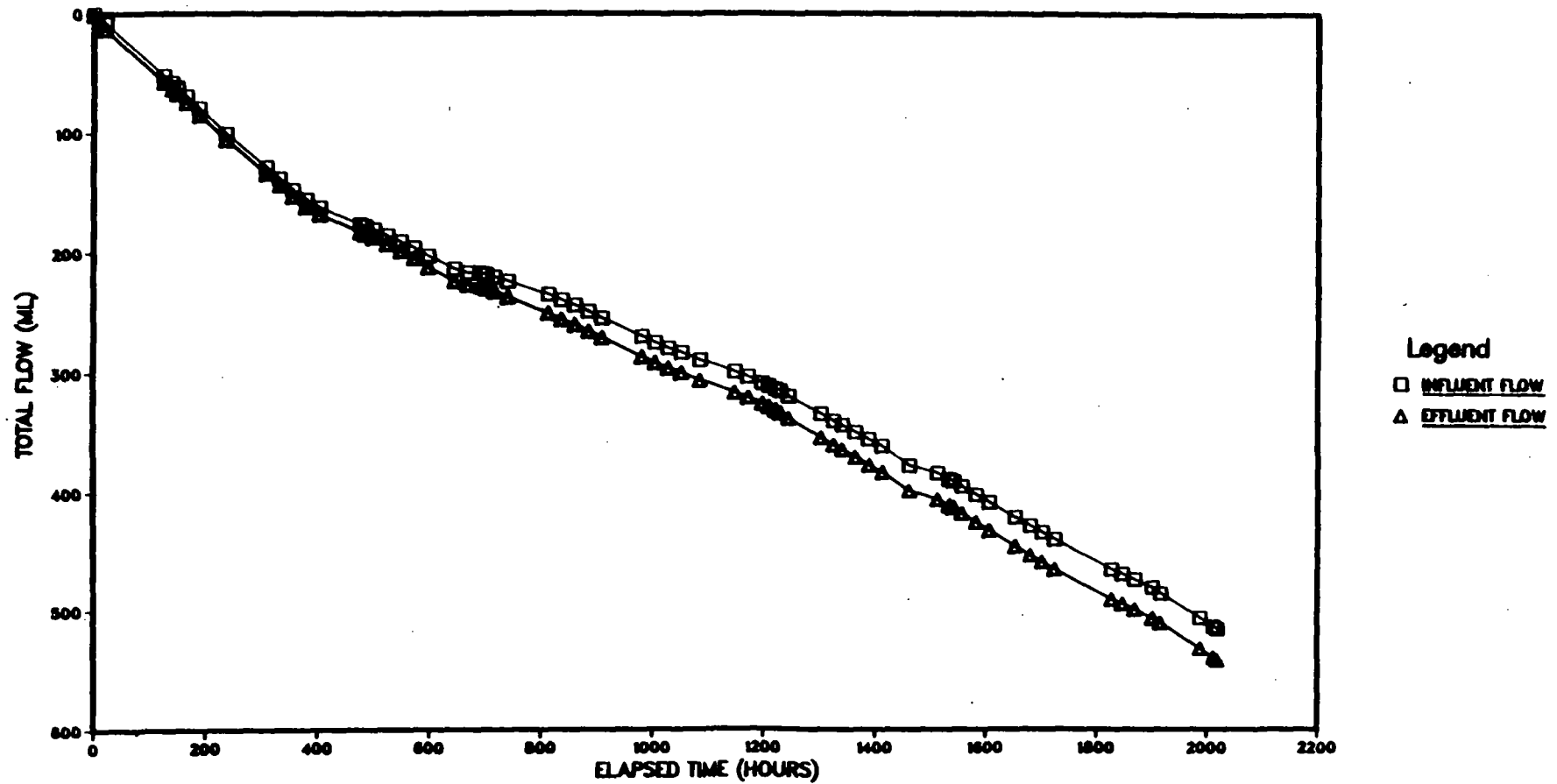
By FAH   Date 3/23/87 Subject CID-AREA 4 -215 P - Samp 3 Sheet No. ____ of ____

Chkd. By DMA Date 3-23-87   PERMEABILITY COMPUTATIONS   Proj. No. CH36-039

1/4" X 1/4"

DEPTH: 14.5 - 15.0'

SAMPLE LENGTH, L = 9.30 cm

SAMPLE DIAMETER = 7.20 cm

CROSS - SECTIONAL AREA, A = 40.72 cm²

INITIAL TIME, t_1 = 910.20 hrs

FINAL TIME, t_2 = 1967.15 hrs

ELAPSED TIME, Δt = 1056.95 hrs

VOLUME OF INFLUENT = 452.70 - 239.30 = 213.40 ml (ml = cm³)

VOLUME OF EFFLUENT = 420.00 - 210.60 = 209.40 ml (ml = cm³)

AVERAGE = 211.4 cm³

VOLUMETRIC FLOW RATE, $Q = \dfrac{\delta V}{\delta t} = \dfrac{211.4 \, cm^3}{1056.95 \, hrs} \cdot \dfrac{1}{\frac{1 \, hr}{3600 \, sec}} = 5.56 \times 10^{-5} \dfrac{cm^3}{sec}$

PRESSURE DIFFERENTIAL, ΔP = 10

DIFFERENTIAL HEAD, $H = \dfrac{\Delta P}{\gamma} = \dfrac{\left(\frac{\#}{in^2}\right)\left(144 \frac{in^2}{ft^2}\right)\left(30.48 \frac{cm}{ft}\right)}{62.4 \frac{\#}{ft^3}}$

$H = 703.4 \, cm$

COEFFICIENT OF PERMEABILITY, $K = \dfrac{QL}{AH}$

$K = \dfrac{\left(5.56 \times 10^{-5} \frac{cm^3}{sec}\right) \cdot (9.30 \, cm)}{(40.72 \, cm^2) \cdot (703.4 \, cm)}$

$\underline{K = 1.8 \times 10^{-8} \, cm/sec}$

LABORATORY PERMEABILITY TEST RESULTS
WMI–CID AREA 4 – BORING 215P SAMPLE 3

Legend
□ INFLUENT FLOW
△ EFFLUENT FLOW

CanonieEnvironmental

Canonie
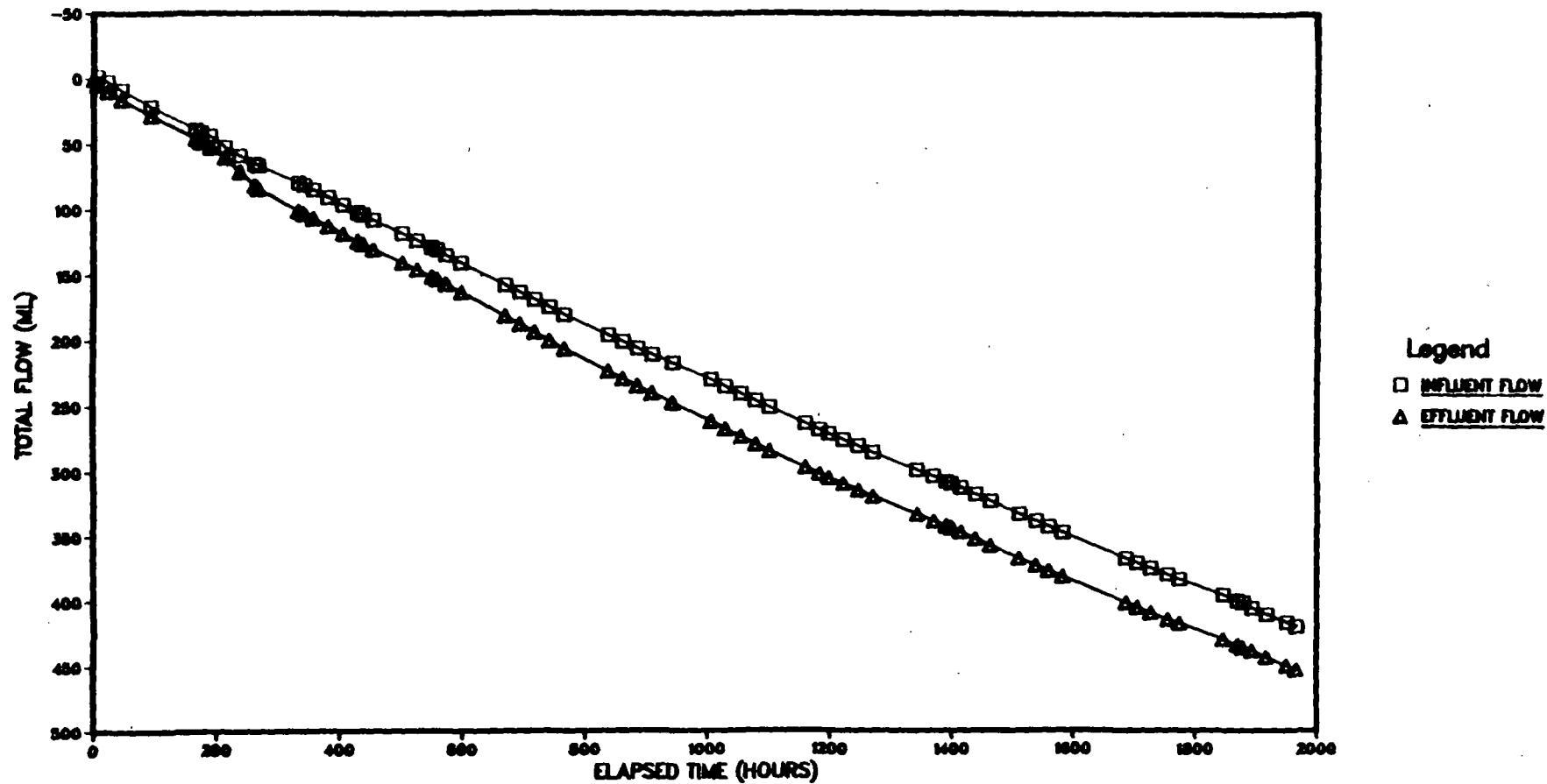
By FAH   Date 3/23/87 Subject CID-AREA4-2½P-Samp4 Sheet No. ___ of ___

Chkd. By DMA Date 3-23-87   PERMEABILITY COMPUTATIONS   Proj. No. CH 86-099

1/4" X 1/4"

SAMPLE LENGTH, $L = 5.70 \text{ cm}^3$

SAMPLE DIAMETER $= 3.50$

CROSS·SECTIONAL AREA, $A = 9.62 \text{ cm}^2$

INITIAL TIME, $t_1 = 1118.03$

FINAL TIME, $t_2 = 1549.63$

ELAPSED TIME, $\Delta t = 431.55 \text{ hr}$

VOLUME OF INFLUENT $= 152.30 - 113.10 = 34.80 \text{ ml}$

VOLUME OF EFFLUENT $= 171.30 - 132.50 = 39.30 \text{ ml}$

AVERAGE $= 37.05 \text{ ml} \approx 37.05 \text{ cm}^3$

VOLUMETRIC FLOW RATE, $Q = \dfrac{\delta V}{\delta t} = \dfrac{39.55 \text{ cm}^3}{431.55 \text{ hr}} \cdot \dfrac{1}{\frac{1 \text{ hr}}{360}} = 2.38 \times 10 \text{ cm}^3$

PRESSURE DIFFERENTIAL, $\Delta P = 10 \text{ psi}$

DIFFERENTIAL HEAD, $H = \dfrac{\Delta P}{\gamma} = \dfrac{\left(\frac{lb}{in^2}\right)\left(144 \frac{in^2}{ft^2}\right)\left(30.18 \frac{cm}{ft}\right)}{62.4 \frac{lb}{ft^3}}$

$H = 703.4 \text{ cm}$

COEFFICIENT OF PERMEABILITY, $K = \dfrac{QL}{AH}$

$K = \dfrac{\left(2.55 \times 10^{-5} \frac{cm^3}{sec}\right)(5.70 \text{ cm})}{(9.62 \text{ cm}^2)(703.4 \text{ cm})}$

$K = 2.0 \times 10^{-8} \text{ cm/sec}$

LABORATORY PERMEABILITY TEST RESULTS
WMI-CID AREA 4 - BORING 216P SAMPLE 4

Legend
□ INFLUENT FLOW
△ EFFLUENT FLOW

TOTAL FLOW (ML)

ELAPSED TIME (HOURS)

CanonieEnvironmental
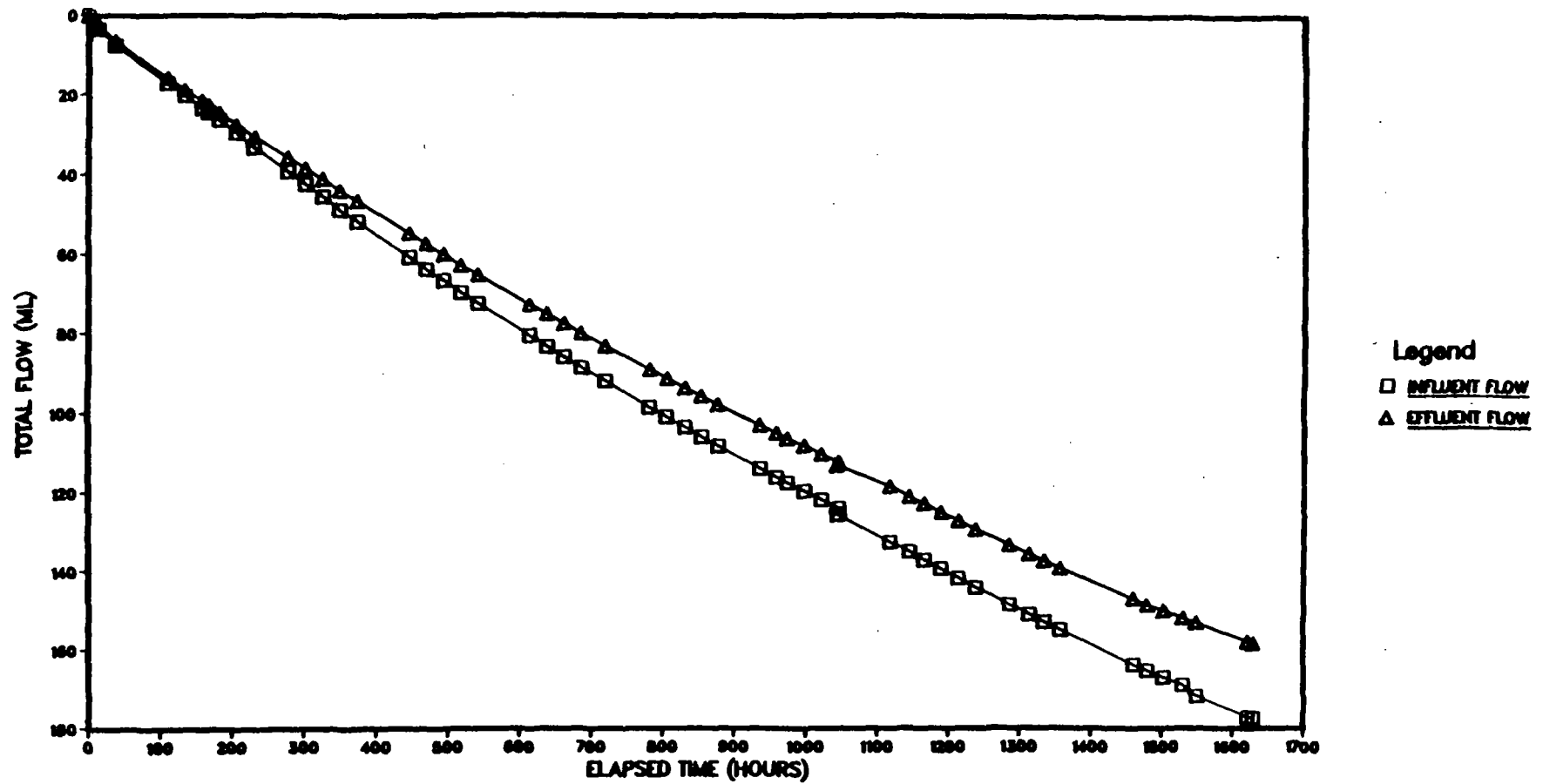
REFERENCES

1. Csallany, Sandor, and Walton, W. C., 1963, Yields of Shallow Dolomite Wells in Northern Illinois, Illinois State Water Survey Report of Investigation 46.

2. D'Appolonia, D. J., 1980, Soil-Bentonite Slurry Trench Cutoffs, Journal of Geotechnical Engienering Division.

3. Hughes, G. M., Kroatz, P., and Landon, R. W., 1966, Bedrock Aquifers of Northeastern Illinois, Illinois State Geological Survey Circular 406.

4. Hughes, G. M., Landon, R. A. and Farvolden, R. N., 1969, Hydrogeologic Data from Four Landfills in Northeastern Illinois, Illinois State Geological Survey Environmental Geology Note 26.

5. Hughes, G. M., Lnadon, R. A. and Farvolden, R. N., 1971, Summary of Findings on Solid Waste Disposal Sites in Northeastern Illinois, Illinois State Geological Survey Environmental Geology Note 45.

6. Hughes, G. M., Schleichner, J. A., and Cartwright, K., 1976, Supplement to the Final Report on the Hydrogeology of Solid Waste Disposal Sites in Northeastern Illinois, Illinois State Geological Survey Environmental Geology Note 80.

7. Larsen, J. I., 1973, Geology for Planning in Lake County, Illinois, Illinois State Geological Survey Circular 481.

8. Lineback, J. A., Gross, D. L., and Meyer, R. P., 1974, Glacial Tills under Lake Michigan, Studies of Lake Michigan Bottom Sediments II, Illinois State Geological Surey Environmental Geology Note 69.

9. Prickett, T. A., et al., 1964, Groundwater Development in Several Areas of Northeastern Illinois, Illinois State Water Survey Report of Investigation 47.

10. Suter, M., et al., 1959, Preliminary Report on Ground-Water Resources of the Chicago Region, Illinois, Cooperative Groundwater Report 1, Illinois State Water Survey and Illinois State Geological Survey.

11. Visocky, A. P., Sherrill, M. G., and Cartwright, K., 1985, Geology, Hydrology and Water Quality of the Combrian and Ordivician Systems in Northern Illinois. Cooperative Groundwater Report 10, Illinois State Geological Survey and Illinois State Water Survey.

12. Walton, W. c., 1965, Ground-Water Recharge and Runoff in Illinois, Illinois Sate Water Survey Report of Investigation 48.

13. Woller, D. M. and Gibb, J. P., et al., 1976, Public Ground-Water Supplies in Lake County, Illinois State Water Survey, Bulletin 60-20, Urbana, Illinois.

SUMMARY OF FINDINGS ON SOLID WASTE DISPOSAL SITES IN NORTHEASTERN ILLINOIS

G. M. Hughes, R. A. Landon, and R. N. Farvolden

SUMMARY OF FINDINGS ON SOLID WASTE
DISPOSAL SITES IN NORTHEASTERN ILLINOIS

G. M. Hughes, R. A. Landon, and *R. N. Farvolden*[†]

INTRODUCTION

The landfill is the most commonly used approved method of solid waste disposal, and has replaced the open burning dump in most areas. A sanitary landfill is defined by the American Society of Civil Engineers as "...a method of disposing of refuse on land without creating nuisances or hazards to public health or safety by utilizing the principles of engineering to confine the refuse to the smallest practical area, to reduce it to the smallest practical volume, and to cover it with a layer of earth at the conclusion of each day's operation, or such more frequent intervals as may be necessary." (Am. Soc. Civil Engrs., 1959, p. 1). The definition implies that if a landfill is truly a "sanitary landfill" it will not adversely affect the quality of surface or ground water.

The results and conclusions of an investigation of the hydrogeology and geochemistry of five landfills (fig. 1) in northeastern Illinois are summarized here. The study was made to develop guidelines that could be used to evaluate the pollution potential of existing and proposed landfill sites. The investigation was supported in part by the Solid Waste Management Office, U. S. Environmental Protection Agency (formerly the Bureau of Solid Waste Management, U. S. Public Health Service, Department of Health, Education and Welfare), Grant no. G06-EC-00006. It was conducted mainly by personnel of the Illinois State Geological Survey and was sponsored by the Survey, the Illinois Department of Public Health, and the University of Illinois. The comprehensive

[*] Present address—Moody and Associates, Inc., Meadville, Pa.

[†] Present address—Department of Earth Sciences, University of Waterloo, Waterloo, Ontario, Canada.

- 1 -

TABLE 1—U. S. PUBLIC HEALTH SERVICE DRINKING WATER STANDARDS AND COMPOSITION OF VARIOUS LIQUID WASTES
(In parts per million)

Substance	U. S. Public Health Service standards[a] Group I[a,b,c]	Group II[d,e]	Leachate Blackwell[f]	LW5B Du Page[g]	LW6B Du Page[h]	Influent sewage[i]	Effluent sewage[i]	Slaughter-house wastes[j]	Chemical plant effluent[k]
Alkyl benzene sulfonate	0.5	•		0.72	0.30				
Arsenic	0.01	0.05	4.31	< 0.10	4.6				
Chloride	250		1,697	1,330	135			520	1,070
Copper	1		0.05	< 0.05	< 0.05	0.450	0.032		2.1
Carbon chloroform extract	0.2								
Cyanide	0.01	0.2	0.024	< 0.005	0.02		0.051		
Fluoride		3.4		2	0.31				800
Iron	0.3		5,500	6.3	0.6	2.600	0.938		51
Manganese	0.05		1.66	0.06	0.06				0.48
Nitrate	45		1.70	0.70	1.60				864
Phenols	0.001								
Sulfate	250		680	2	2			370	8,120
Total dissolved solids	500		19,144	6,794	1,198			2,690	16,090
Zinc	5			0.13	< 0.10	0.638	0.366		
Barium		1	8.5	0.80	0.30				
Cadmium		0.01	< 0.05	< 0.05	< 0.05	0	0		
Chromium (Cr^{+6})		0.05	0.20	0.15	< 0.05	0	0		
Lead		0.05		0.50	0.50	0.158	0.158		
Selenium		0.01	2.7	< 0.10	< 0.10				
Silver		0.05	< 0.1	< 0.1	< 0.1				
Ammonium						19	16		198
Alkalinity (as CaCO$_3$)			3,255	4,159	1,011			440	760
Hardness (as CaCO$_3$)			7,830	2,200	540				0
Phosphate			6	1.20	8.90			66	74
Titanium									0.97
Aluminum			2.20	0.10	0.90				6.4
Sodium			900	810	74				6,190
Hexane solubles			550	18	7	22.4	11		
Biological oxygen demand[l]			54,610	14,080	225	104	17	3,700	
Chemical oxygen demand			39,680	8,000	40	240	70	8,620	
pH				6.3	7.0	7.2	7.4	8.1	6.2

[a] U. S. Department of Health, Education and Welfare (1962).
[b] Nitrates exceeding 45 ppm dangerous for infants.
[c] Should not be used if more suitable supplies available.
[d] Larger concentrations should be rejected.
[e] Fluoride is temperature dependent.
[f] Probably represents leachate from compaction and infiltration.
[g] Leachate from refuse about 6 years old.
[h] Leachate from refuse about 17 years old.
[i] Data provided by Metropolitan Sanitary District of Greater Chicago.
[j] Data from files of the Illinois Department of Public Health
[k] Rare earth and thorium production (Butler, 1965, p. 65).
[l] Twenty-day biological oxygen demand for leachate. Other values are 5-day BOD.
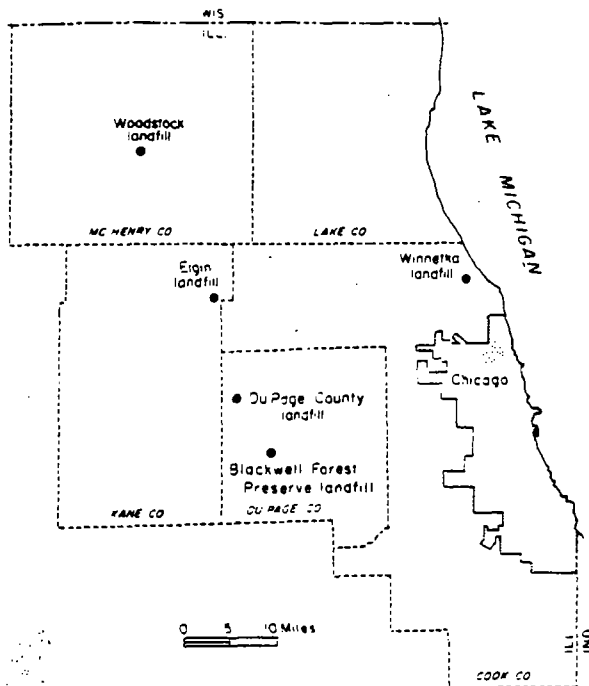
Fig. 1 - Locations of landfill sites investigated.

report for this investigation is being published by the Solid Waste Management Office. Some of the data from the study were published by the Geological Survey in 1969 (Hughes, Landon, and Farvolden).

For the investigation, well points and piezometers were installed around and below existing landfills to determine the pattern of ground-water flow, and samples of ground water were gathered from selected points and analyzed. Data were collected on the composition of dissolved solids in water draining from the refuse (leachate) and on the attenuation of the various dissolved solids in the leachate as it moved away from the disposal site.

The data suggest that large areas of northeastern Illinois could be used for solid waste disposal without affecting the ground-water resource. Methods of designing and operating landfill facilities in various hydrogeologic environments also were suggested by the findings of the study.

FINDINGS OF PREVIOUS INVESTIGATIONS

Previous investigations of landfills in Europe and the United States have brought out the following facts.

1. A leachate capable of polluting ground and surface water is commonly produced by refuse in contact with water. (The composition of such leachate is compared with that of other liquid wastes in table 1.) The water that leaches the refuse may be ground water or infiltrating precipitation. In an arid climate, such as that of southern California, precipitation is not adequate to infiltrate and produce a leachate from buried refuse. Refuse buried above the top of the zone of saturation in such areas will not endanger ground-water resources unless the area is flooded. In a wet climate, such as is found in Britain, however, investigations have shown that precipitation will infiltrate refuse and produce leachate.

2. Dissolved solids in leachate travel with the ground water and may, under certain circumstances, so degrade the ground water that it can no longer be used for domestic purposes.

3. Gases, predominantly methane and carbon dioxide, also are produced by the decomposition of refuse. Methane may cause explosions, and carbon dioxide may increase the hardness of the ground water.

Top of saturated zone
nearly coincides with
land surface

Recharge   Discharge   Recharge   Discharge   Recharge

Glacial till   Dolomite aquifer

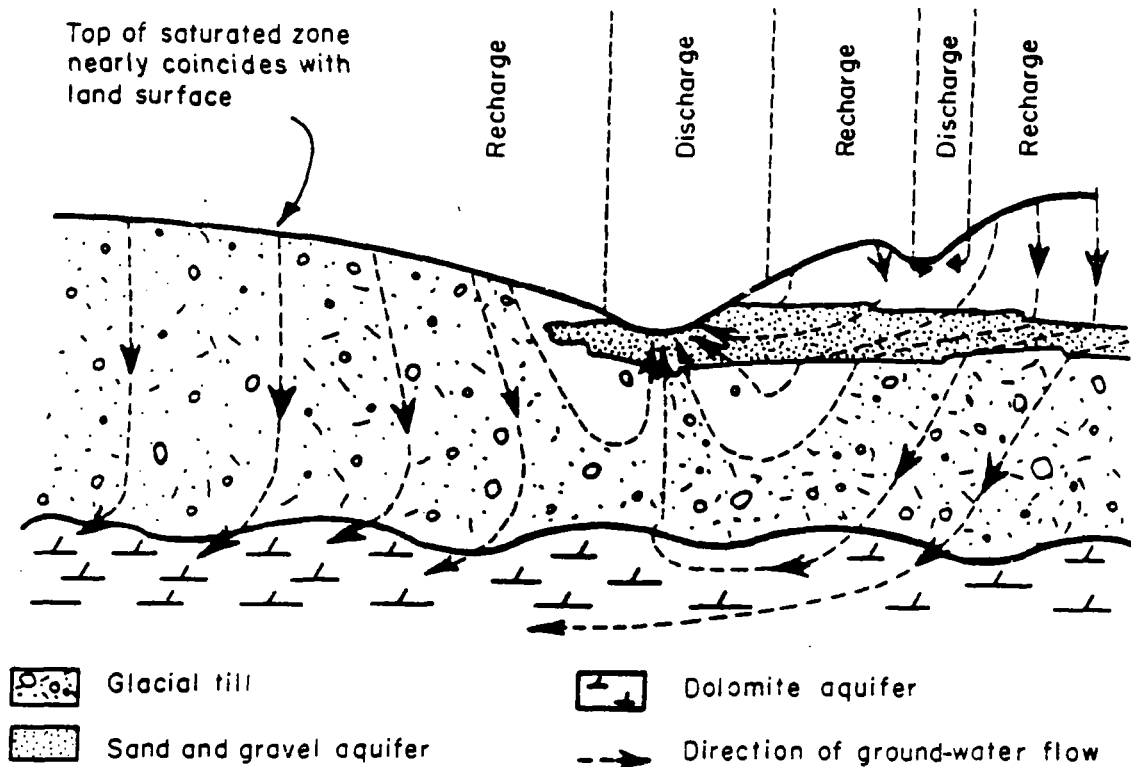Sand and gravel aquifer   Direction of ground-water flow

Fig. 2 - Hypothetical system of ground-water flow.

4.   The length of time required for refuse to stabilize and cease
producing contaminants cannot readily be predicted. The process is depen-
dent upon a number of factors, including the moisture available, the temper-
ature, the materials present in the landfill, and probably upon the conditions
of burial and compaction. Some landfills stabilize in a few years; others
still produce methane after 30 years.

METHODS OF INVESTIGATION

A ground-water flow system is the path water takes through the earth.
In the subsurface of a humid region such as northeastern Illinois, the top of
the zone of saturation, or water table, is close to the land surface. Ground
water occupies all the openings in the earth material below the water table.
Above the water table the openings are filled with both water and air. A part
of the precipitation or other water that has entered the ground moves downward
to the water table and enters (recharges) the ground-water flow system. This
water moves through the ground to a point of discharge at the earth's surface,
such as a stream, swamp, marsh, or lake (fig. 2).

To trace contaminants from a disposal site into the surrounding earth
materials, it was necessary to delineate, both quantitatively and qualitatively,
the ground-water flow system operating at the site. This involved determining

how much water was going into the landfill, how much leachate was leaving the landfill, and what paths the water and leachate followed.

Five sites were studied during our investigation. Existing sites were chosen because the leachate leaving them can be traced, and because landfills of various ages provide a record of changes in the composition of the leachate over a considerable time period. They also were in environments typical of those likely to be used for future landfills.

During the investigation, 274 piezometers and sampling points were installed at the five sites. A piezometer is a screen or permeable plastic tip fastened to the end of a pipe or tube. The pipe is installed in a boring, and the opening around the pipe above the tip is sealed with bentonite. Water enters the pipe from the ground through the tip. Water-level measurements in the pipe and water samples recovered from the pipe are used to determine the pressure and quality of water in the restricted zone around the piezometer. Water from higher in the boring cannot enter the pipe because it is sealed off by the bentonite. A well point is similar to a piezometer, except that there is no seal to restrict water movement around the outside of the pipe and, therefore, measurements or water samples obtained from a well point may reflect conditions throughout a large vertical interval instead of at a particular point.

Water analyses were performed by the Illinois Department of Public Health, commercial laboratories, and the Illinois State Geological Survey. The Survey also made the analyses of the earth materials associated with the landfills. Monthly, weekly, and continuous hydrographs of water levels were compiled, together with records of barometric changes and precipitation. Permeability data were gathered from tests performed in the field and in the laboratory.

RESULTS OF INVESTIGATION

Old Du Page County Landfill

The old Du Page County landfill is located in the NW¼ Sec. 32, T. 40 N., R. 9 E., Du Page County. It is in a flat upland area that was originally swampy but was drained through tiles into Kress Creek, which flows to the south along the eastern side of the landfill area. The general sequence of earth materials in the area consists of an upper surficial sand, 10 to 20 feet thick, overlying approximately 55 feet of silty and sandy clay till that contains one thin, interbedded sand unit. The till overlies the dolomite bedrock, which is a major aquifer in this area.

Filling by the trench and fill method began in 1952 and was completed in 1966. Some of the trenches intersected the top of the zone of saturation. The refuse was piled to a maximum thickness of 20 feet. The final fill cover was 2 to 3 feet thick and consisted primarily of silt loam, clay, silty clay loam, and clay loam.
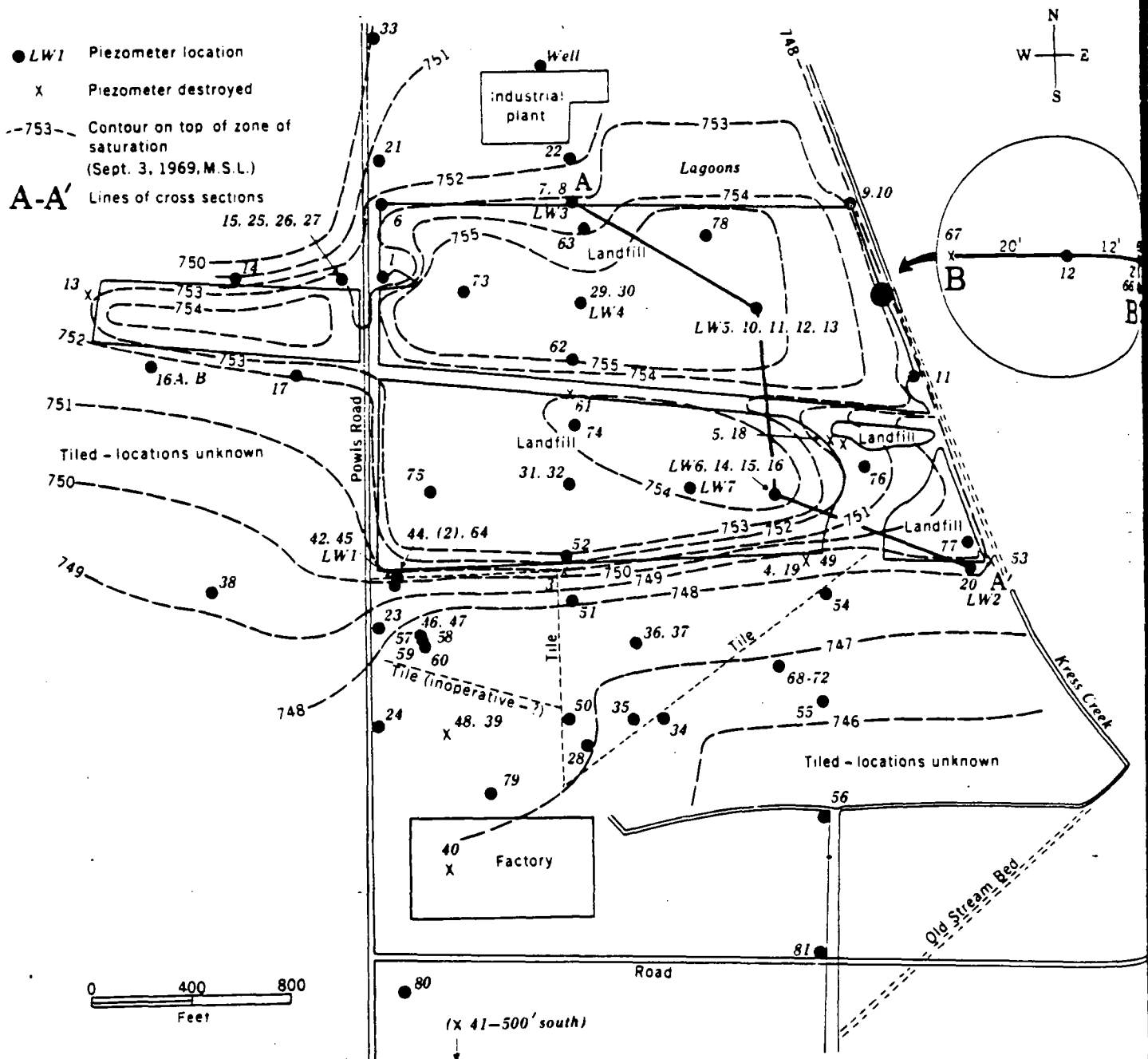
Fig. 3 - Plan view of the old Du Page County landfill showing locations of borings and the top of the zone of saturation.

Figure 3 is a plan view of this landfill and the surrounding area, showing the location of the borings and the contours of the top of the zone of saturation. A ground-water mound 6 feet high has developed in the landfill, and ground-water movement is away from the landfill in all directions.

Ground-water mounds have also formed beneath two of the other disposal sites studied. Such mounds form where infiltrated precipitation is restricted from free lateral flow by the surrounding earth materials at the margins of the landfill. Commonly the ground water is discharged as springs or seeps at the edge of the landfill, where the ground-water mound intersects land surface.

Figure 4 shows vertical sections across the filled area, its lithology, and the equipotential lines. Movement is predominantly lateral but is somewhat downward through the surficial sand. There is a nearly vertical gradient downward through the underlying till. Section B-B' shows the influence of Kress Creek on the configuration of the flow system along the east side of the landfill.

Of the 28.58 inches of rain that fell on this landfill from October 1, 1968, through September 30, 1969, approximately 15.6 inches infiltrated the landfill. Based on the area involved, this is a rate of 90,000 gallons per day. We calculate that 87 percent of this infiltrated water moved laterally out of the landfill through the surficial sands and 13 percent moved downward through the till beneath the landfill.

Figure 5 presents chloride concentrations in the leachate in the landfill and in the surficial deposits surrounding the landfill. Chloride is perhaps the most mobile and easily detected of the ions in the leachate and therefore is an excellent tracer. Chlorides have moved at least 600 feet, but not more than 900 feet, southward from the landfill. This distance agrees with calculations of the velocity of ground-water movement through the sand. Our data indicate that the biological oxygen demand, the chemical oxygen demand, and the potassium and iron values of the leachate were reduced by approximately two orders of magnitude by the time the leachate had traveled approximately 600 feet south of the old Du Page landfill. Hardness, sodium, calcium, and bromine were reduced by approximately one order of magnitude, and other components of the leachate were reduced by various degrees. Sulfate, phosphate, and nitrate were the only components that showed a definite increase in concentration with distance away from the landfill. Their increase is attributed to the fact that these components cannot exist in the reducing environment caused by the high organic content of leachate, but, as the organic components are attenuated away from the fill and reducing conditions become weaker, the nitrate, sulfate, and phosphate radicals can exist.

Data from a series of wells completed in the upper part of the till beneath the Du Page landfill show that in leachate moving a distance of 4 to 5 feet through the till the chloride content and the total dissolved solids content decreased more than one order of magnitude and the organic material decreased two orders of magnitude. Dissolved solids from the landfill were not detected in the interbedded sand 20 feet below the top of the glacial till, in the dolomite bedrock, or in the creek adjacent to the landfill.
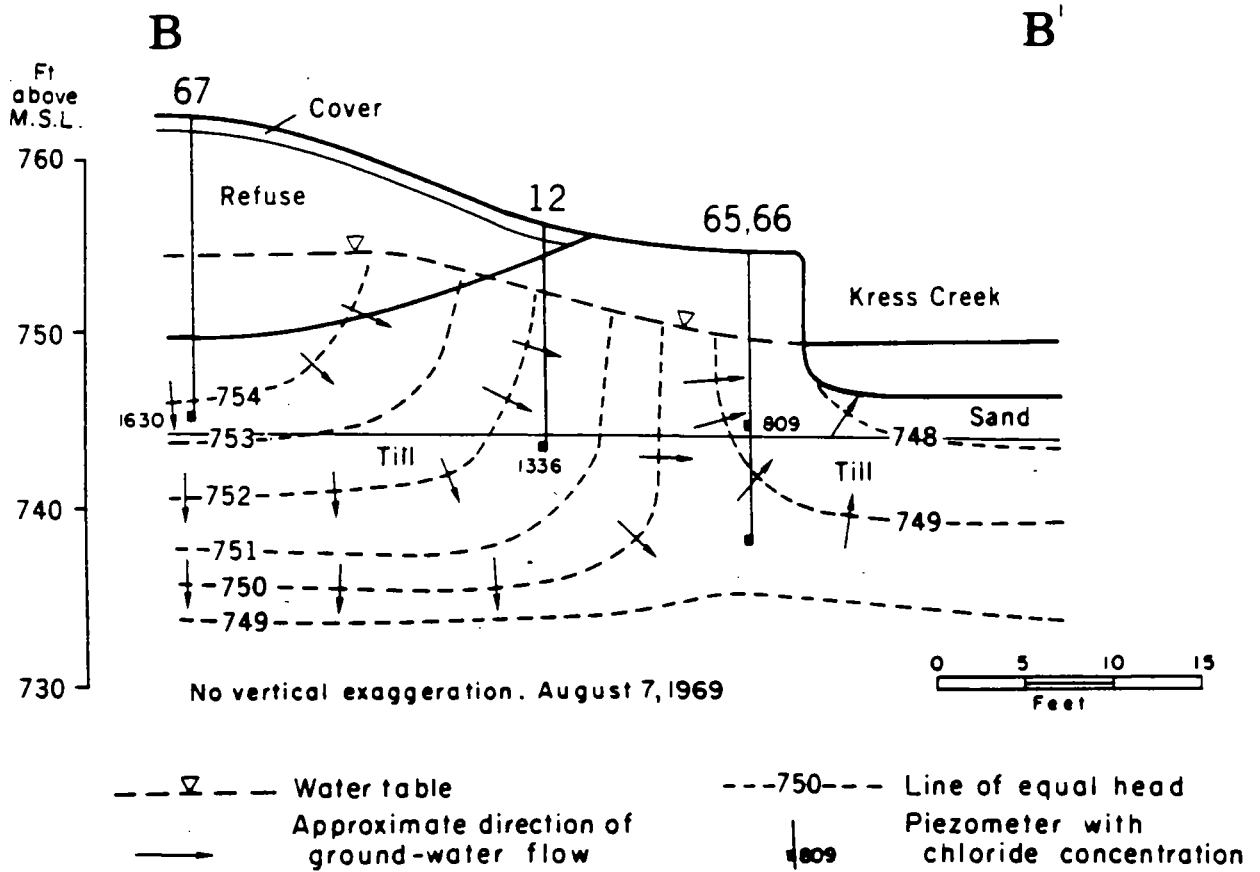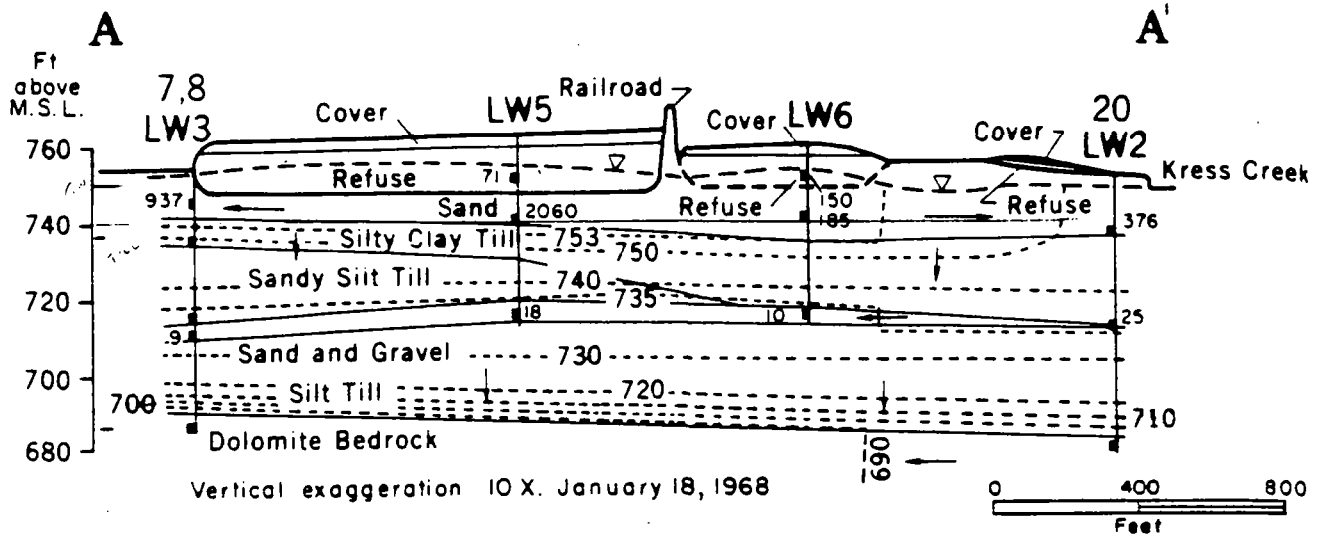
Fig. 4 - Cross sections of the old Du Page County landfill. Selected chloride concentrations and lines of equal head are shown.
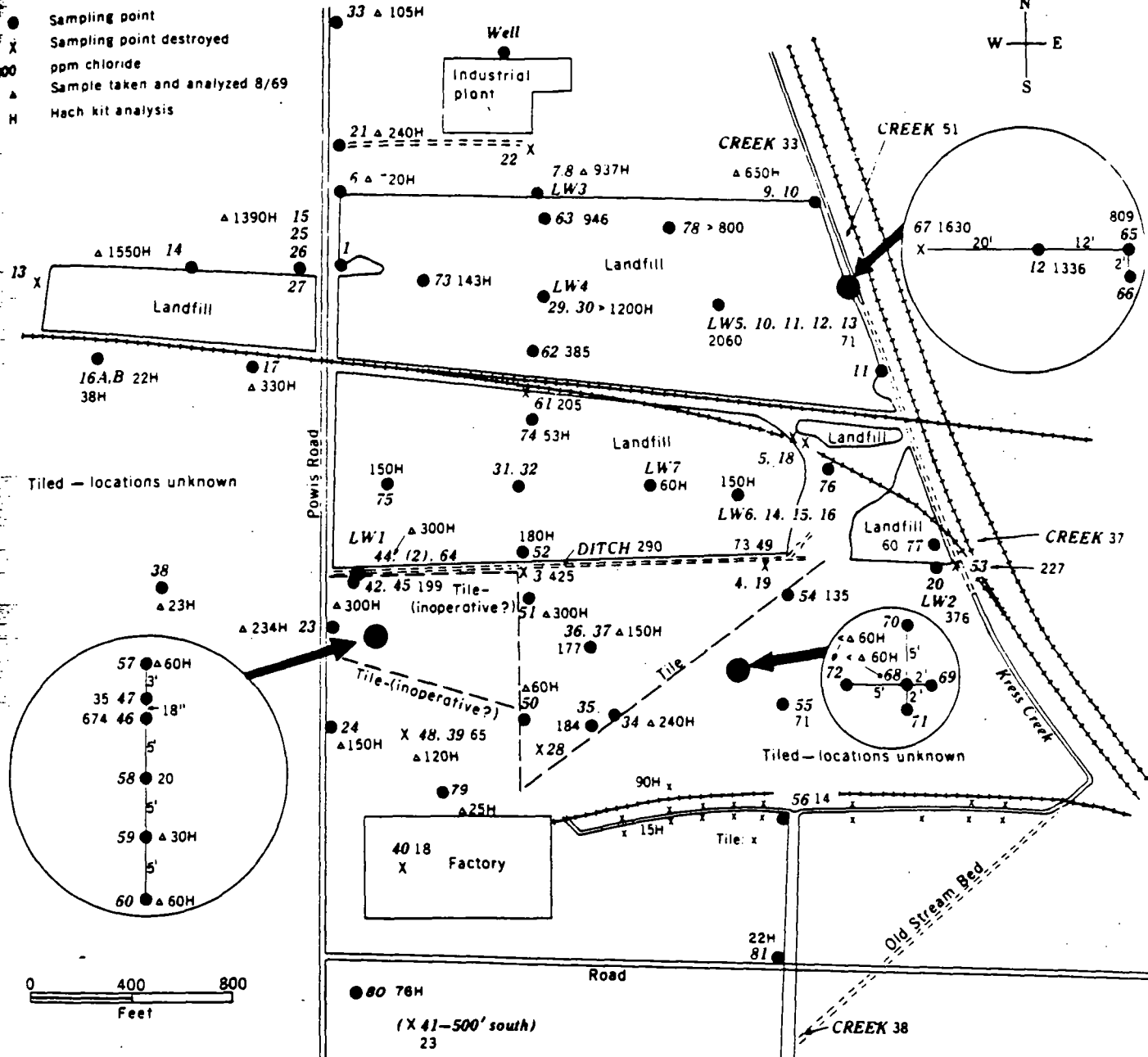
Cr ek

● Sampling point
X Sampling point destroyed
300 ppm chloride
▲ Sample taken and analyzed 8/69
H Hach kit analysis

N
W — E
S

33 ▲ 105H

Well

Industrial plant

21 ▲ 240H
22 X

CREEK 33

CREEK 51

6 ▲ ⁻20H
7.8 ▲ 937H
LW3
9. 10 ▲ 650H

▲ 1390H 15
25
26
27
▲ 1550H 14
13 X
Landfill

63 946
78 > 800

Landfill

67 1630
X 20'   12 1336   12' 65 809
2'
66

73 143H
LW4
29. 30 > 1200H

LW5. 10. 11. 12. 13
2060   71

62 385

11

16A,B 22H
38H
17
▲ 330H

61 205
74 53H

Landfill
Landfill

5. 18 X

CREEK 37

Tiled — locations unknown

Powis Road

150H
75
31. 32
LW7
60H
150H
LW6. 14. 15. 16

76

Landfill
60 77
53
227

38
▲ 23H

LW1
44. (2). 64 ▲ 300H
42. 45 199 Tile-
▲ 300H   (inoperative?)
▲ 234H 23

180H
52
DITCH 290
73 49

51 ▲ 300H
36. 37 ▲ 150H
177

4. 19
20
54 135
LW2
376

Tile

70
▲ 60H
< ▲ 60H 5'
72   68 2' 69
5'   2'
71

57 ●▲ 60H
3'
35 47
674 46   18"
5'
58 ● 20
5'
59 ● ▲ 30H
5'
60 ●▲ 60H

Tile-(inoperative?)

▲ 60H
50
35
184
34 ▲ 240H

24
▲ 150H
X 48. 39 65
▲ 120H
X 28

55
71

Tiled — locations unknown

79
▲ 25H

90H X
15H
56 14
Tile: X

40 18
X   Factory

22H
81

Old Stream Bed

Kress Creek

0   400   800
Feet

Road

● 80 76H

(X 41—500' south)
23

CREEK 38

Fig. 5 - Selected chloride concentrations in surficial sand and gravel at the old
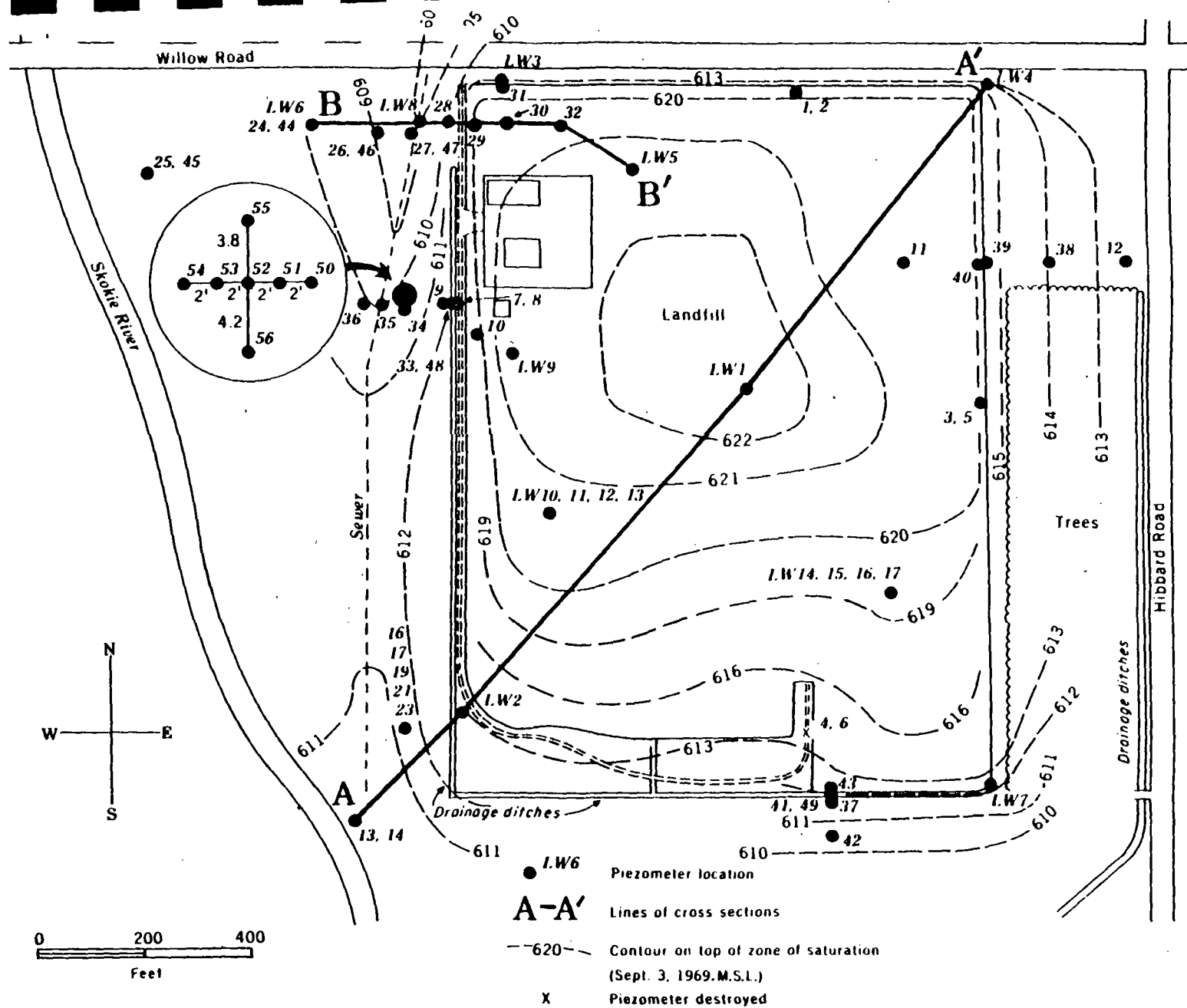Du Page landfill.

Fig. 6 – Plan view of the Winnetka landfill showing locations of borings and the top of the zone of saturation.

Apart from the springs along the side of this landfill, which could probably be considered as no more than a local nuisance, this site has had little effect on the surrounding environment. The ground water in the shallow surficial sand surrounding the landfill has been degraded to some extent, but the sand is not considered an aquifer. Neither the deeper aquifers underlying the landfill nor the creek along the east side of the landfill have been measurably affected.

Winnetka Landfill

The Winnetka landfill is located in the SE¼ Sec. 19, T. 42 N., R. 13 E., Cook County. The topography is quite flat and the landfill itself is the highest point in the vicinity. Geologic materials present in the area consist of 5 to 11 feet of sandy clay and silt alluvium, underlain by approximately 100 feet of silty clay and sandy till that contain thin interbedded sand and silt stringers. The underlying bedrock is a fractured dolomite aquifer.

Filling was begun in January 1947 and the landfill is still operating. The cover is 1 to 3 feet thick and consists mainly of clay loam and sandy loam. Refuse was dumped in trenches 5 to 6 feet deep, which intersected the top of the zone of saturation. The refuse was piled 6 to 8 feet above the original land surface.

Figure 6 is a plan view of the Winnetka landfill and the surrounding area, showing the location of the borings and contours of the top of the zone of saturation. As at the Du Page County landfill, a ground-water mound 8 to 10 feet high has formed beneath the filled area, and leachate springs are present along the edge of the landfill. The slope on the west side of this mound is quite steep because water is draining into a sewer. Figure 7 shows two cross sections of the fill area, indicating flow through the surficial alluvium and a downward gradient through the underlying till. Lenses of sand and silt within the till section are not shown because they cannot be correlated from boring to boring.

The location of a sewer on the west side of the filled area is shown on cross section B-B' (fig. 7). This sewer distorts the flow system and serves as a collector for part of the water moving out of the west side of the landfill.

Of the 35.20 inches of rain that fell from October 1, 1968, to September 30, 1969, approximately 15.6 inches infiltrated the landfill. Based on the area involved, this is a rate of 28,300 gallons per day. We calculate that approximately 94 percent of this water moved laterally through the alluvium and 6 percent moved downward through the till beneath the landfill.

Figure 8 shows the chloride concentration in water from the surficial alluvium in the vicinity of the Winnetka landfill. Dissolved solids have reached sampling point 12 approximately 270 feet east of the landfill, but they have been intercepted by the sewer and ditches along the west and south sides of the landfill. There is some evidence that dissolved solids from this landfill have migrated downward from the landfill through approximately 19 feet of
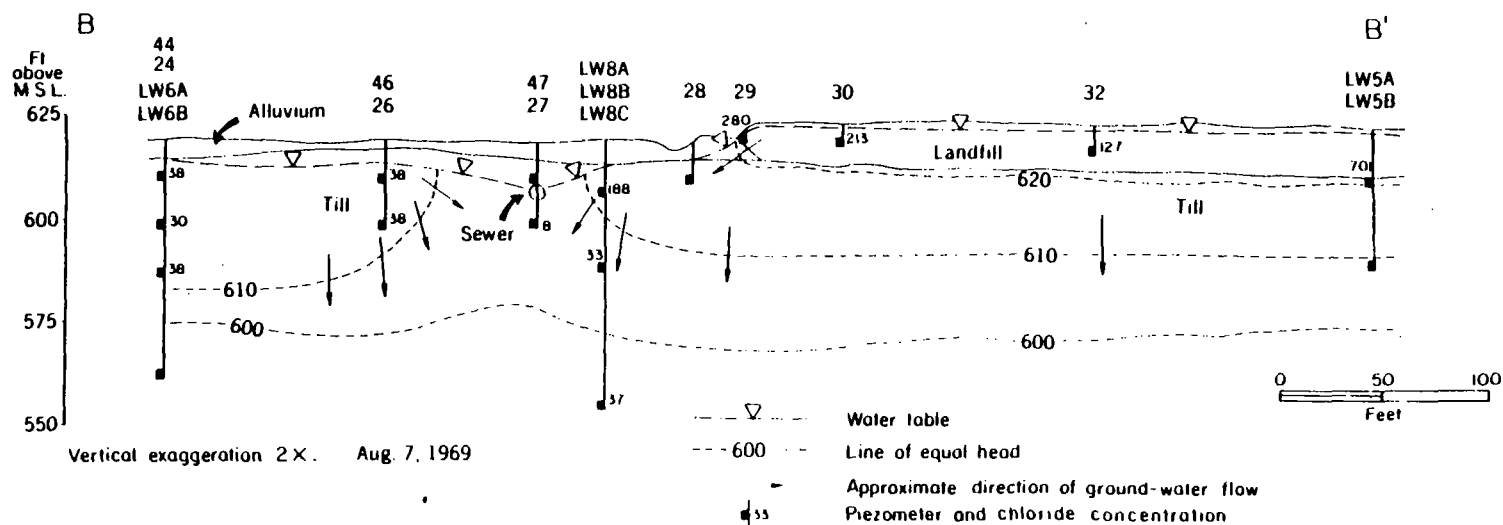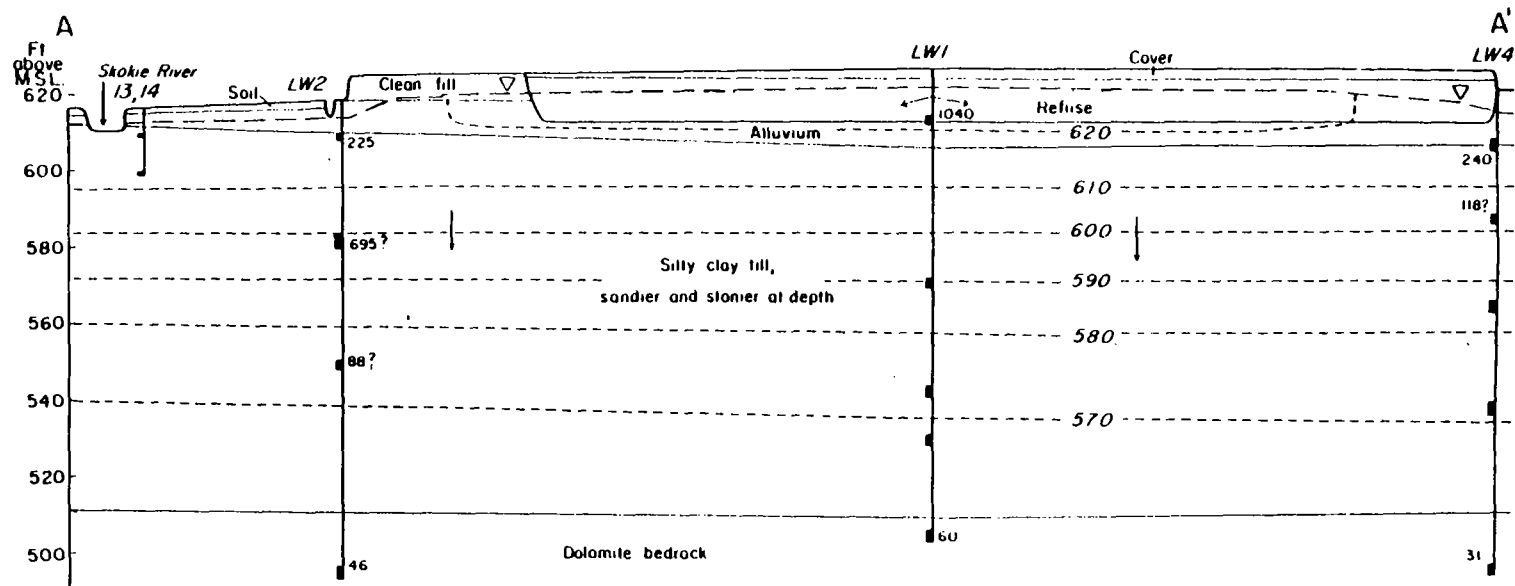
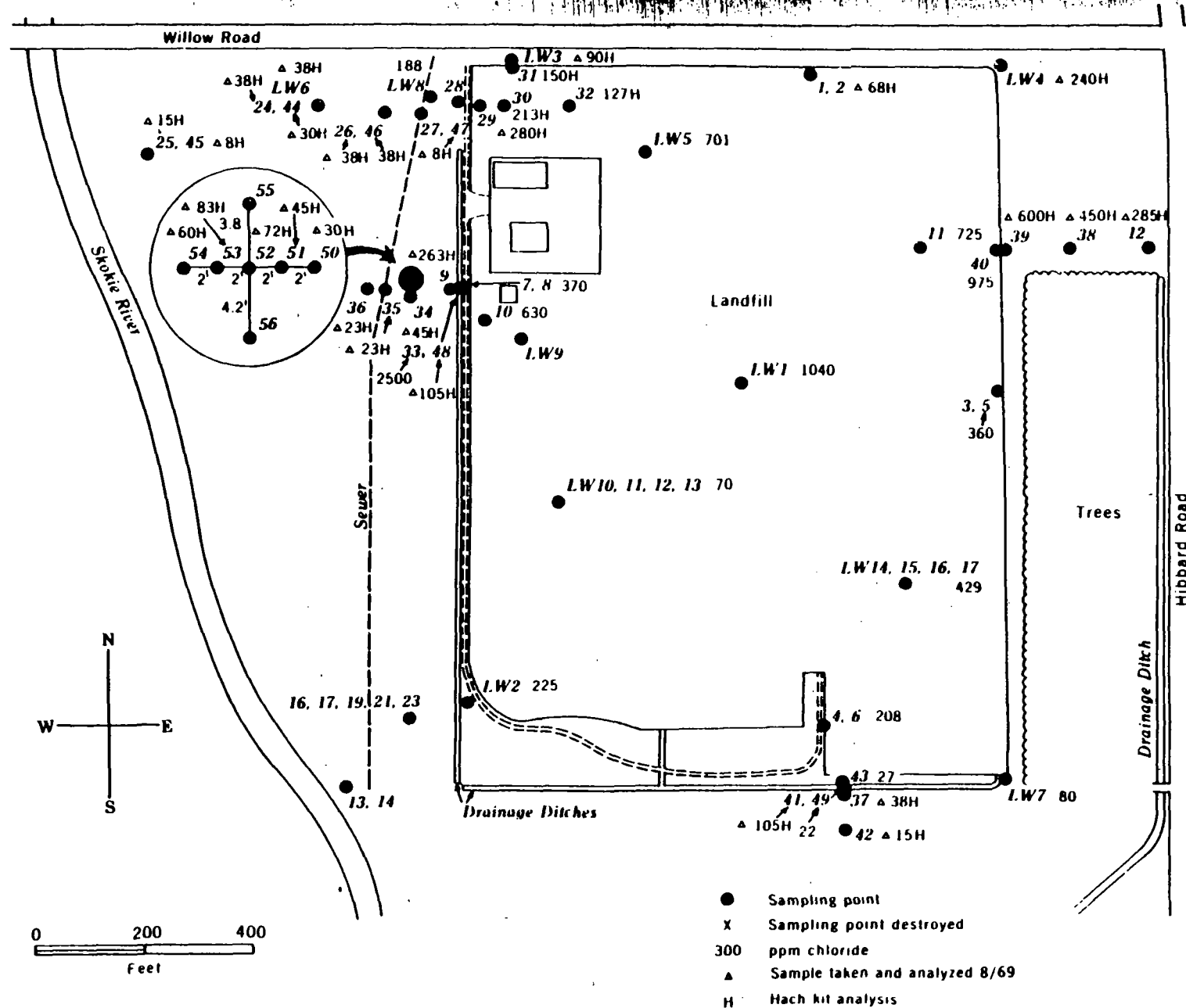Fig. 7 — Cross sections of the Winnetka landfill and selected chloride concentrations.

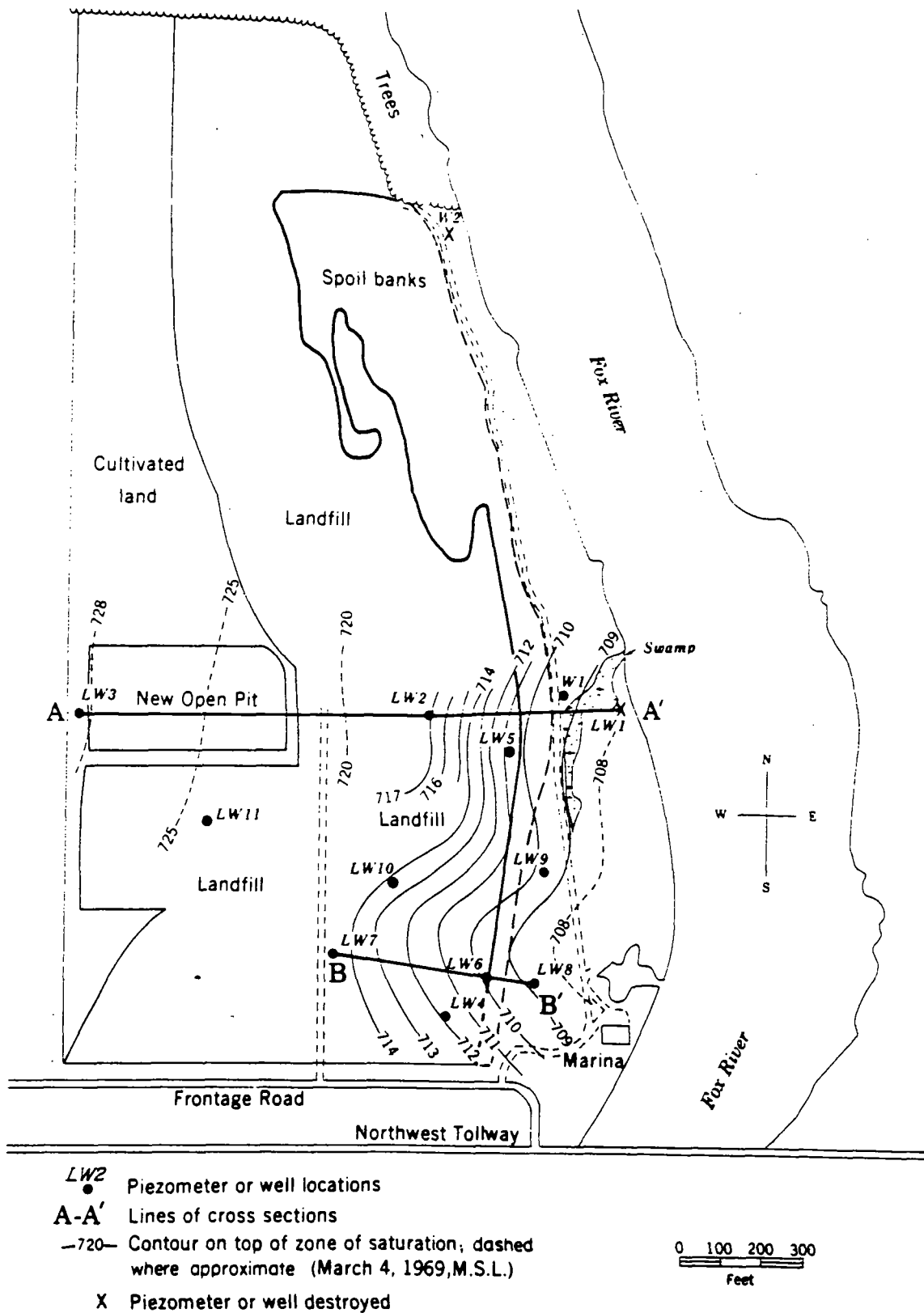Fig. 8 - Selected chloride concentrations in the alluvium at the Winnetka landfill.

Fig. 9 - Plan view of the Elgin landfill showing locations of borings and the top of the zone of saturation.

alluvium and 14.5 feet of glacial till. The till in this area contains stringers of sand and silt, particularly in its upper part, which may account for the depth reached.

The Winnetka landfill has had very little effect on the surrounding environment. Although surface seepage is present around the edges of the landfill, it has not caused serious problems, and there is no evidence that leachate has moved down far enough to pollute the underlying dolomite aquifer.

Elgin Landfill

The Elgin landfill is located in the SW¼ Sec. 35, T. 42 N., R. 8 E., Kane County. It is on the west side of the Fox River Valley, and the ground surface slopes to the Fox River on the east. The geologic materials surrounding this site consist of up to 20 feet of coarse-textured sand and gravel, overlying approximately 15 feet of sandy silt till. The till, in turn, overlies 2 to 5 feet of sand and gravel above fractured dolomite bedrock.

The site was a gravel pit before filling began in 1948. Initially, it was used as an open burning dump, but in 1964 it was converted into a sanitary landfill of the trench and fill type. In parts of the area some refuse was emplaced slightly below the top of the zone of saturation. The cover is loam and clay loam approximately 2 feet thick.

Figure 9 is a plan view of the Elgin landfill that shows the location of borings and the contours on top of the zone of saturation. There is no evidence of a ground-water mound at this site, and the water table slopes relatively smoothly to the east and southeast towards the Fox River.

Figure 10 shows two cross sections of the landfill. The sections indicate predominantly lateral movement of the ground water and discharge upward into the Fox River. The Elgin landfill is located in the discharge area bordering the Fox River, and, as the Fox River is one of the major drainages in northeastern Illinois, this is probably a major discharge area.

Of the 26.2 inches of rain that fell from October 1, 1968, to September 30, 1969, approximately 15 inches infiltrated the landfill. Based on the area involved, this is a rate of 66,000 gallons per day. All of this water eventually discharges into the Fox River.

Figure 11 shows the water quality determinations in the vicinity of the Elgin landfill. Correlation between distance from the landfill and the water quality is not as good as at the other sites, probably because variations in the permeability of the shallow sands and gravels allow differential movement of the dissolved solids. Dissolved solids from the landfill are moving out of the site to the east through the uppermost sand and gravel deposit into the Fox River. They have not moved to the west or downward through the till beneath the landfill.

Apart from degrading the ground water in the shallow sand and gravel aquifer between the landfill and the river, the Elgin landfill has had little
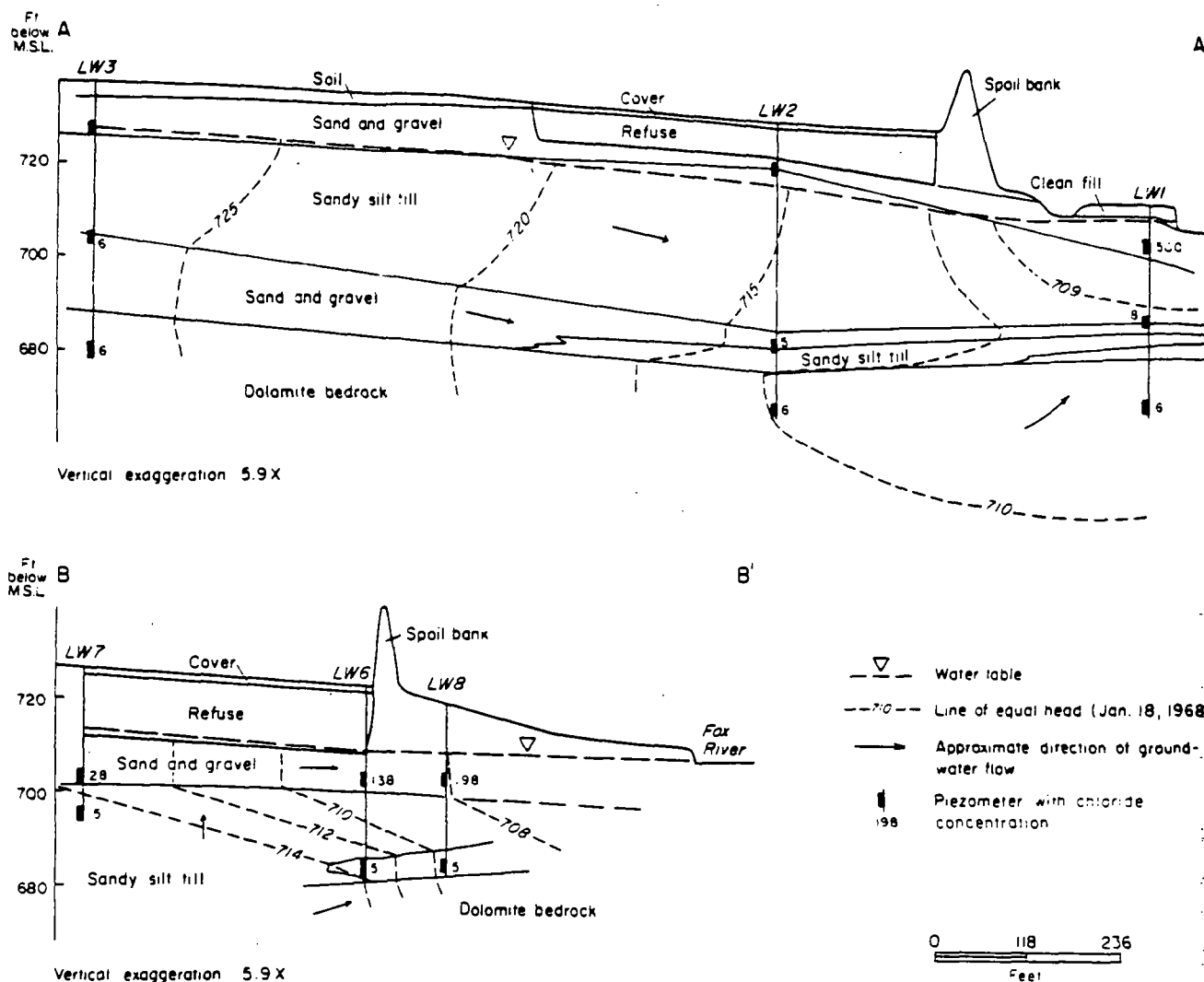
Fig. 10 - Cross sections of the Elgin landfill and selected chloride concentrations.

effect on the surrounding environment, and dissolved solids have not entered
the deeper sand and gravel and dolomite units beneath the sandy silt till.
These deeper units are aquifers and offer a source of shallow ground water.
We estimate the leachate leaving the landfill would raise the dissolved solid
content in the Fox River at the point of discharge by approximately 0.30
part per million, half of which is hardness.

Woodstock Landfill

The Woodstock landfill is located in the NE¼ Sec. 17, T. 44 N., R.
7 E., McHenry County. The topography of the area is morainic. The landfill
lies on the top and south flank of an east-west trending upland and in the
swampy lowland to the south of this upland.

The geologic materials present consist of a sequence of silty clay
till and sandy till interbedded with sand and gravel to a depth of more than
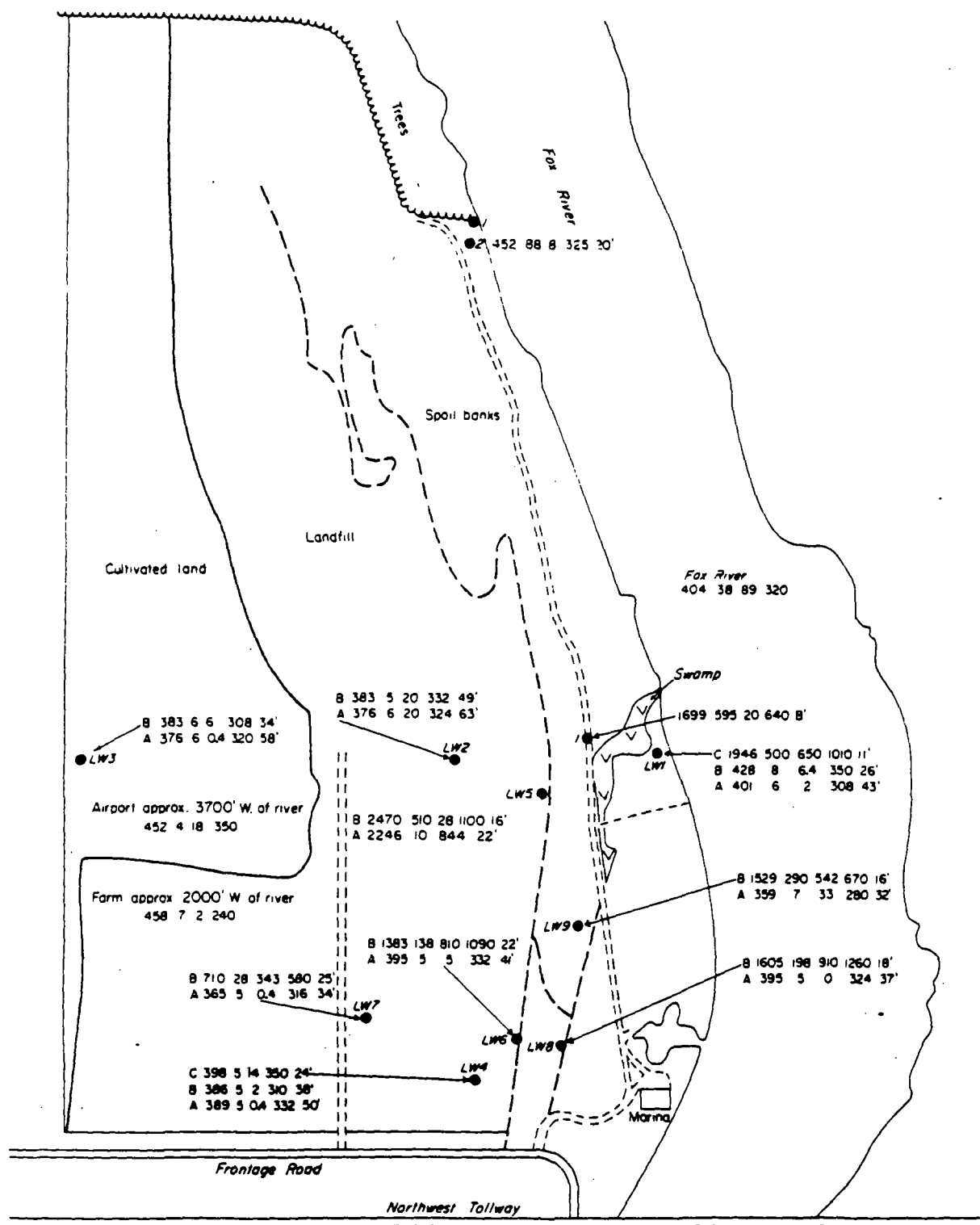
EXPLANATION OF MAP NUMBERS

Sampling point	TDS (ppm)	Cl (ppm)	SO₄ (ppm)	Hardness (ppm)	Depth (or formation) sampled (ft)
LW6 A	261	40	0	108	58'

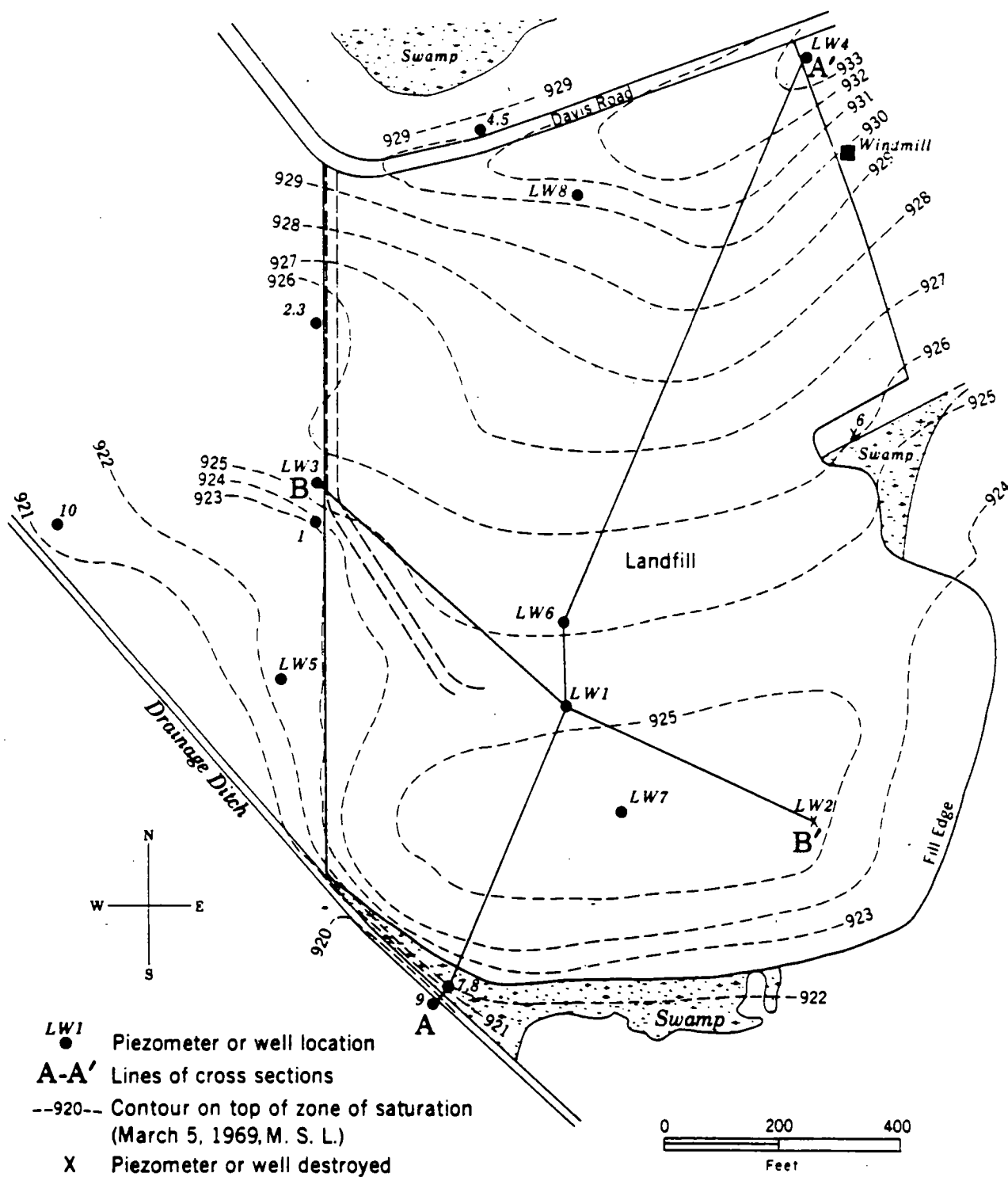Fig. 11 - Water quality data for the Elgin landfill.

Fig. 12 - Plan view of the Woodstock landfill showing locations of borings
and the top of the zone of saturation.

225 feet. In the lowland area, 5 to 19 feet of peat and silt overlie these materials.

The landfill was first operated as an open burning dump, beginning in 1940, and was converted to a sanitary landfill in 1965. The landfill is still in operation. In some parts of the area, refuse was deposited below the top of the zone of saturation. The final cover consists of 2 to 3 feet of loam and silt loam, silty clay loam, and sandy loam.

Figure 12 is a plan view of the Woodstock landfill showing the location of the borings and contours of the top of the zone of saturation. In the northern part of the landfill, gradients on the top of the zone of saturation are away from the upland in all directions. In the southern part of the landfill, the gradient is southward to the swampy areas bordering the landfill or to the drainage ditch west and southwest of the landfill. One effect of the landfill is a steepening of the gradients at the southern edge, which indicates a small ground-water mound lies beneath the landfill.
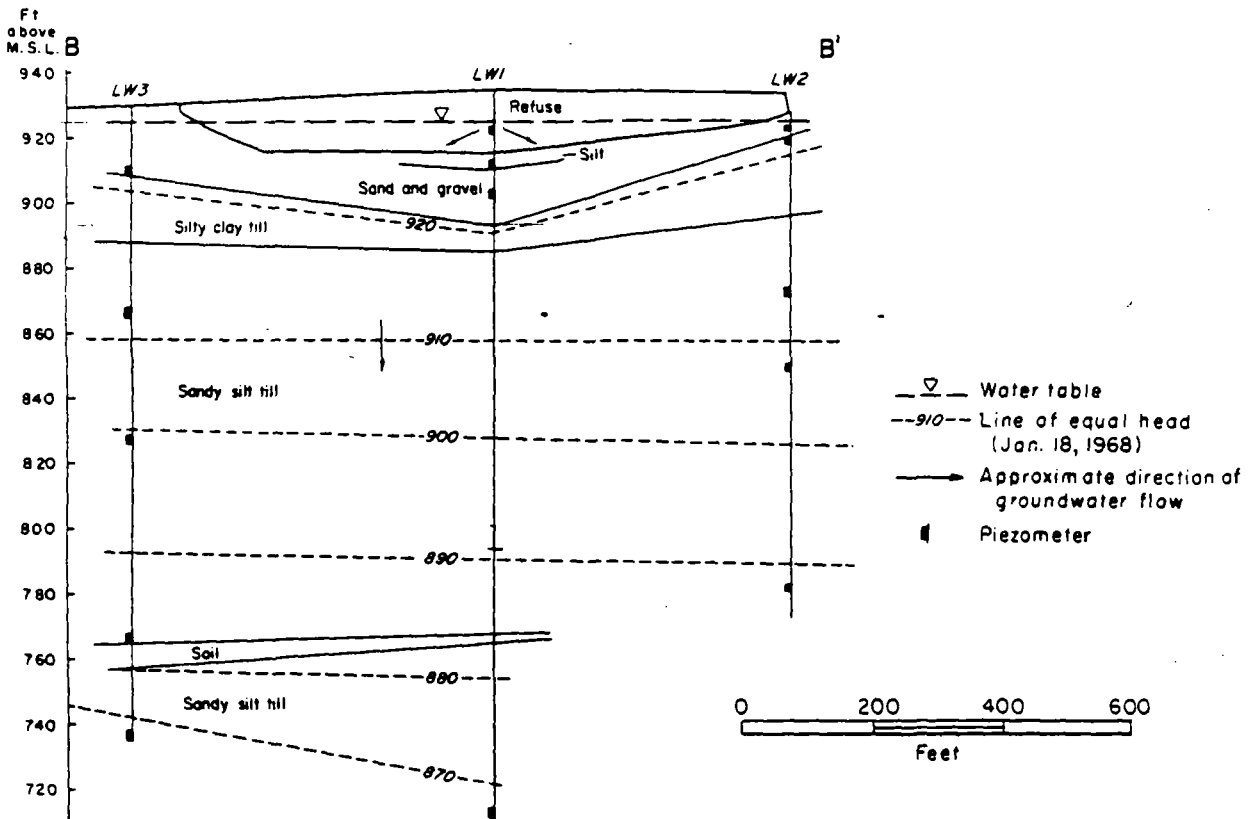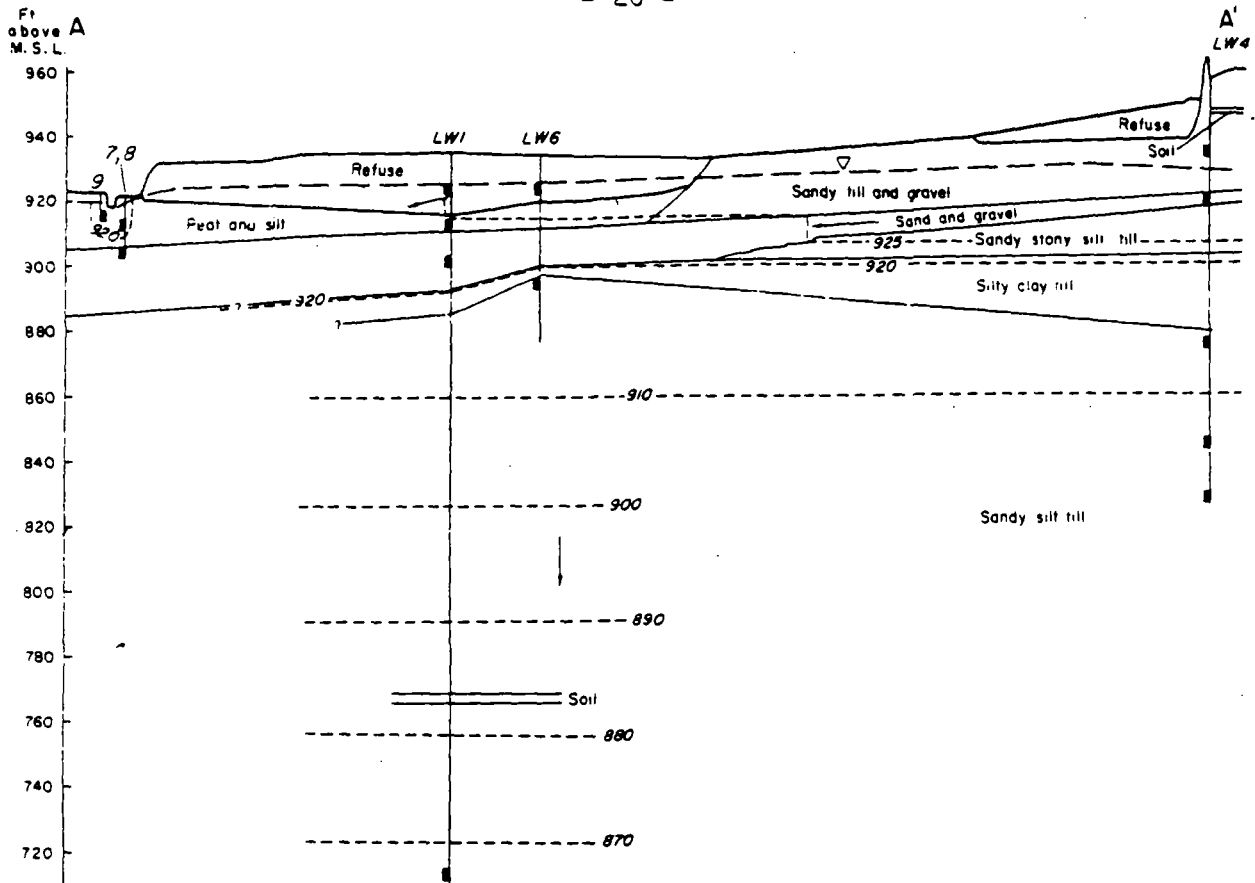
Figure 13 shows two vertical sections across the filled area. There is a strong component of lateral flow in the shallow materials above the silty clay till and a vertical gradient in the silty clay till.

Several interbedded sands and gravels have not been shown on the Woodstock cross sections. These deposits are generally more permeable at Woodstock than at Winnetka and would magnify any horizontal component of the ground-water flow. The drainage ditch west of the landfill area acts in much the same manner as the deep sewer at Winnetka, distorting the flow lines and "collecting" ground water moving from the western side of the landfill.

Of the 24.07 inches of rain that fell from October 1, 1968, to September 30, 1969, approximately 12 inches infiltrated the landfill. Based on the area involved, this is a rate of 22,500 gallons per day. No quantitative evaluation of flow from the Woodstock site was made because of the complex geology and lack of data on the hydrologic properties of the materials.

Water quality data plotted in figure 14 shows the expected inverse relation between the total dissolved solids and the distance from the landfill. There is no movement of dissolved solids downward through the silty clay till. Whether this is because the till has acted as a barrier to their migration, or whether insufficient time has elapsed since the fill was emplaced is not known. Analyses of water in the drainage ditch southwest of the landfill were inconclusive and failed to show whether or not dissolved solids from the landfill have affected this water.

The landfill apparently has had no significant effect on the surrounding environment. The shallow sands and gravels beneath the landfill were degraded by dissolved solids from the landfill; however, the amount of ground water involved is relatively small. After our study had been completed, it was reported that the drainage ditch had been seriously degraded by water moving from the landfill area. Although a detailed investigation was not made, it appeared that, as expansion of the landfill continued to the south, fill material was placed over a broken tile that drained directly into this ditch and that leachate moving through this tile had caused the subsequent problems.

Fig. 13 - Cross sections of the Woodstock landfill.

Vertical exaggeration 5 X

D 583 15 136 540 29'
C 348 10 37 295 73'
B 353 7 11 270 106'
A 343 15 7 250 121'

LW 4

4,5   Davis Road

Windmill
348 8 30 320'

664 9 235 625 20'

2,3

Not filled

417 11 76 375 9'

Ditch south of Davis Rd.
618 100 152 440

F 1314 243 22 650 8'
E 1583 155 15 1010 22'
D 419 4 18 400 65'
C 354 5 25 290 105'
B 404 10 6 310 165'
A 404 6 1 330 195'

LW 3

5

10
583 9 120 540 11'

1545 5 233 1160 9'

Landfill

LW6
A 935 113 7 520 40'

Ditch
478 12 188 465

LW 5

Drainage ditch

C 775 72 360 530 11'
B 427 21 62 310 21'
A 404 7 3 280 47'

LW1
D 6647 2370 345 1000 14'
C 617 80 317 366 25'
B 449 16 87 360 34'

E 371 8 64 360 5
D 377 13 64 272 9'
C 313 4 40 260 57'
B 337 7 12 260 79'
A 346 10 13 270 148'

LW2

Swamp
1646 375 123 830'

3823 728 2000 1550 11'
1492 278 500 900 18'

7,8

695 65 220 570 11'

9

Ditch west side of Rt 47
858 80 398 700'

EXPLANATION OF MAP NUMBERS

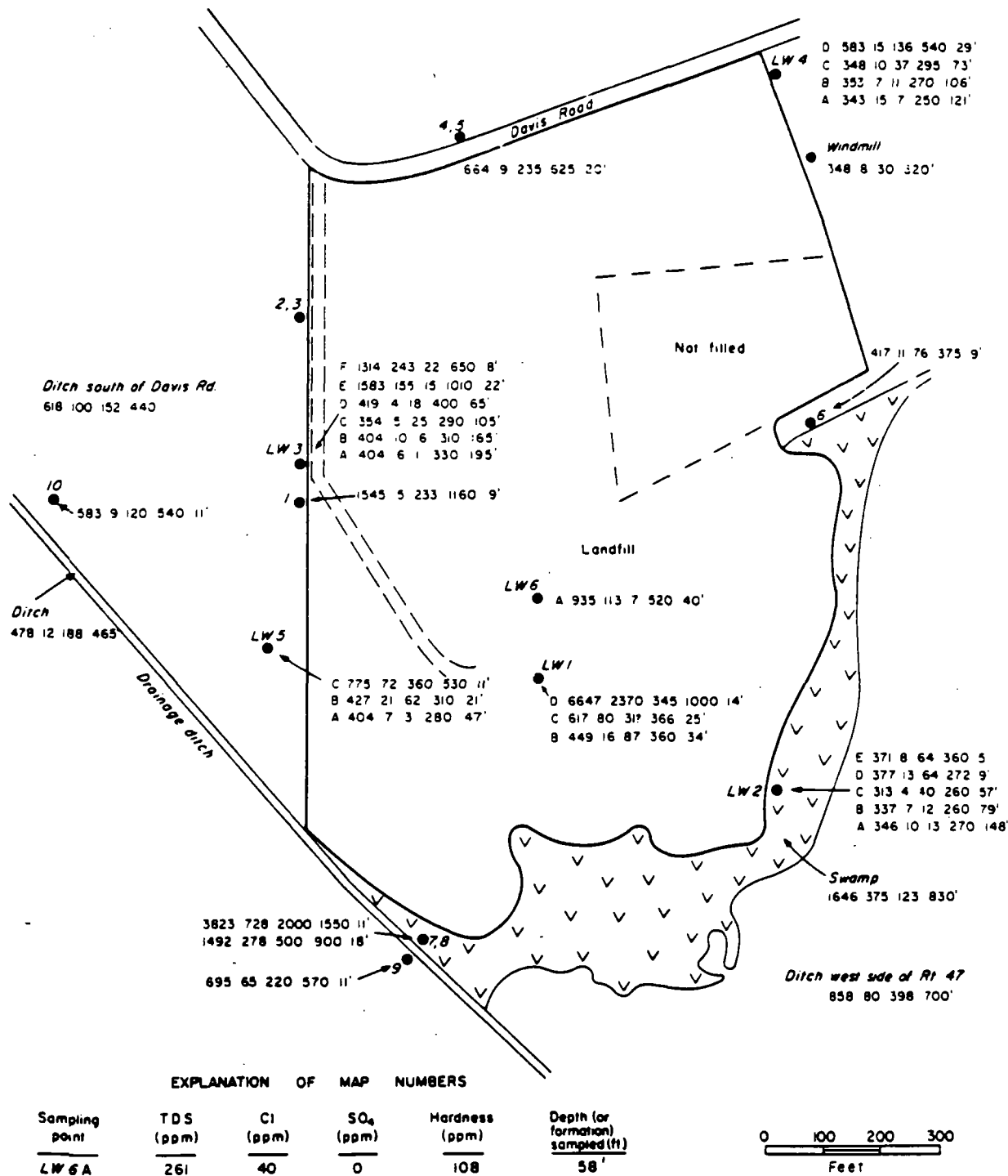| Sampling point	TDS (ppm)	Cl (ppm)	SO₄ (ppm)	Hardness (ppm)	Depth (or formation) sampled (ft)
LW6A	261	40	0	108	58'

0   100   200   300
Feet

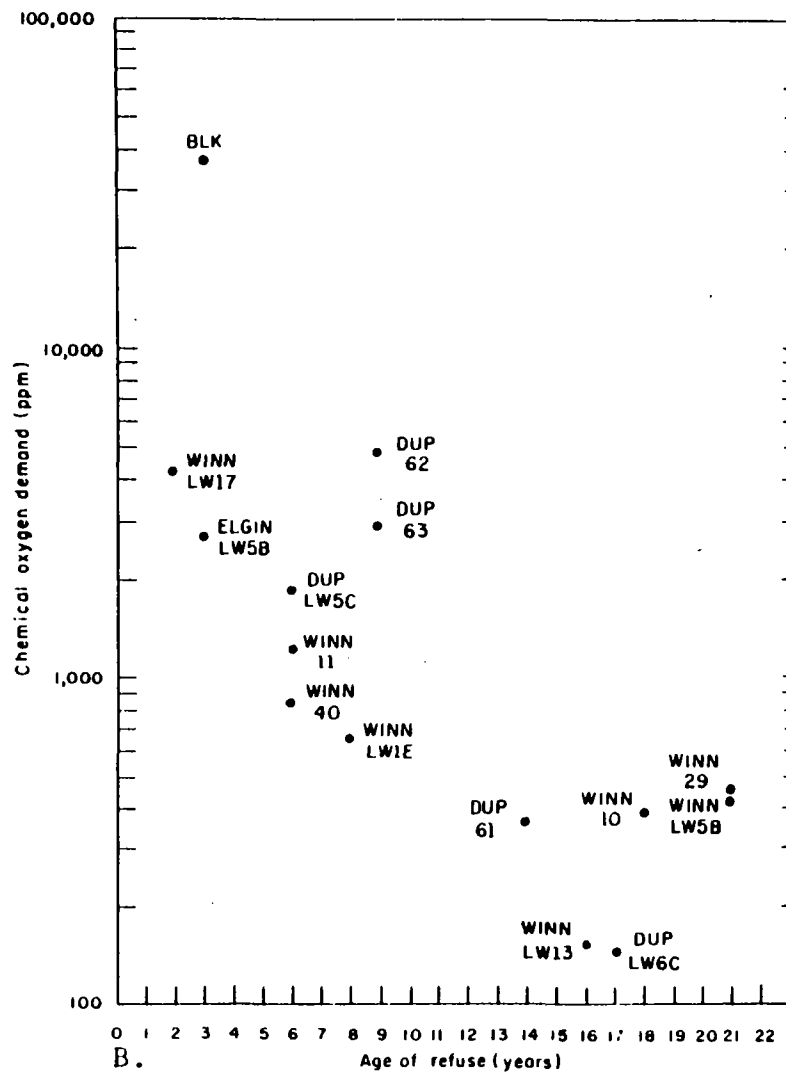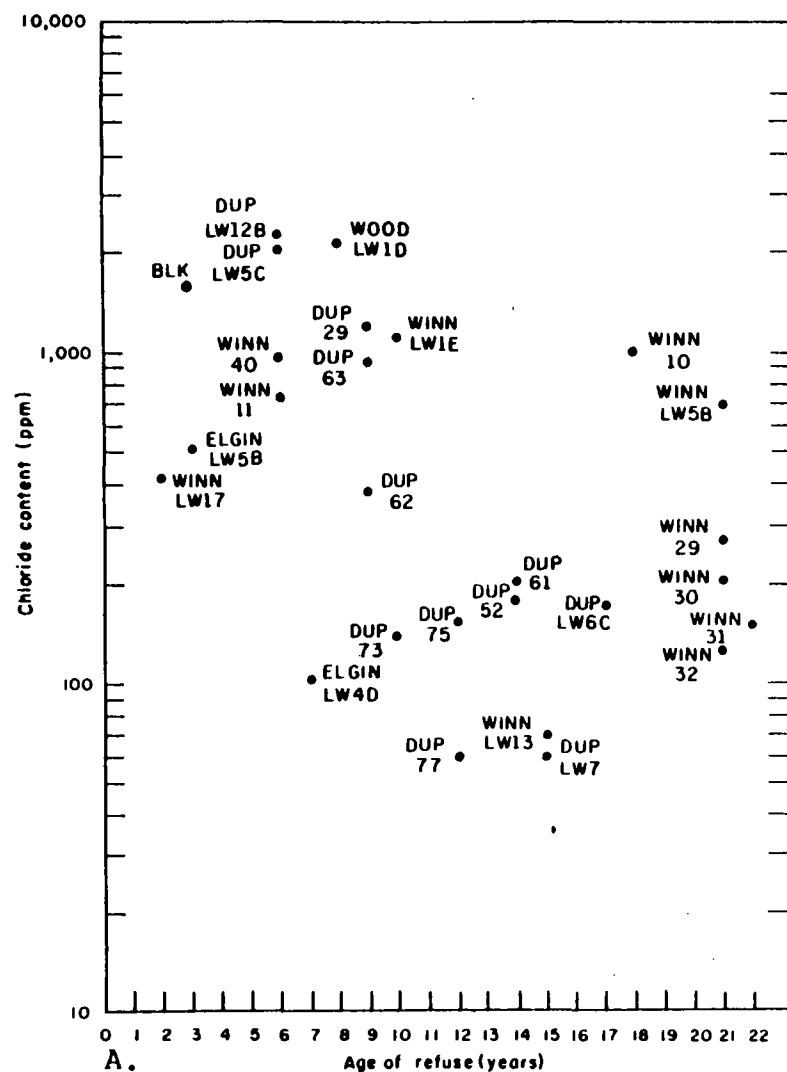Fig. 14 - Water quality data for the Woodstock landfill.

Fig. 15 – (A) Relation between refuse age and chloride content. (B) Relation between refuse age and chemical oxygen demand.

Blackwell Forest Preserve Landfill

The Blackwell Forest Preserve landfill, located near Warrenville in Du Page County, was begun in October 1965 and is to be made into a winter sports hill, which will eventually be about 150 feet high and cover 30 acres. The base of the landfill is lined with 10 feet of silty clay till, and 15-foot berms are being constructed along the sides, which will completely enclose the refuse.

Only one observation well was installed in this landfill. At the time of sampling, when the refuse had been in place for approximately 39 months, the well contained 5 feet of water. This water rises and falls in response to rainfall, and an additional 15 feet of water accumulated during the winter of 1970.

Leachate collected from this well had a biological oxygen demand (20 days) of 54,610 parts per million and a total dissolved solids content of 19,144 ppm. This highly concentrated leachate is typical of that found in relatively young refuse.

Variations in Composition of Leachate with Age of Refuse

Chloride content and chemical oxygen demand of leachate samples are plotted against age of refuse on figures 15A and 15B. The correlation is poor. Much of the scatter can be attributed to normal variations in the composition of the leachate. Refuse more than 20 years old evidently can still have a high content of dissolved solids, indicating that stabilization of landfills is a long process.

Analyses of Landfill Gases

In the course of this study, 20 samples of landfill gases were collected and analyzed for carbon dioxide, oxygen, nitrogen, and methane. The analyses show that gas with a maximum of 84 percent methane is being produced from refuse buried in 1955 at the old Du Page County landfill and that some methane is still being produced from the oldest (1947) part of the Winnetka landfill, indicating that decomposition is still underway.

DESIGN OF LANDFILLS

Landfill sites may be upgraded by various engineering techniques if they do not have naturally protective environments. These techniques may (1) allow the migration of leachate under acceptable conditions, (2) provide for recovery of the leachate, or (3) eliminate the production of leachate altogether.

For any landfill design, the type of earth materials present at the site must be determined and the ground-water flow system known if the landfill is to function properly and if the capacity of the environment for self-purification is to be used advantageously.

Engineering techniques that could be used to make a landfill safe include containment of leachate by installing an impervious lining in the site before the fill is emplaced, reduction of infiltration through the landfill by covering and grading the surface, collection of leachate by tiles or pumping systems, venting of landfill gases, and treatment of leachate.

Other factors that should be considered in landfill design are the type of cover material to be used, the settlement of the fill, the possibility of construction over the completed landfill, and the final use projected for the completed landfill area.

CONCLUSIONS

If ground-water pollution alone is considered, approximately 80 percent of northeastern Illinois would probably be suitable for sanitary landfilling with little or no site modification, because the surficial materials are fine textured, have low permeability, and would restrict the movement of leachate. Another 10 percent of the land area would be suitable because of its favorable location within the hydrogeologic flow system. Sites in the remaining 10 percent of northeastern Illinois may require a considerable amount of modification. Unfortunately, a disproportionately large percentage of the sites proposed as sanitary landfills fall into this last category, a group that includes mined-out quarries and gravel pits. Such sites are easily filled, and, when filled, increase substantially in value. However, they are not safe landfill sites unless modifications are made.

Under typical landfill conditions in northeastern Illinois about half the yearly precipitation will infiltrate the landfill surface. This water, in the form of leachate, runs off on the land surface or enters the ground-water reservoir. If the water infiltrates the fill and moves downward, ground-water mounds are formed. Three of the disposal sites studied had such mounds. Ground-water mounds may result in the formation of springs around the margin of the filled area.

In humid areas requirements that refuse be placed above the top of the zone of saturation are not likely to prevent the production of refuse leachate. Such requirements may in fact lead to locating landfills in less satisfactory environments. For instance, in upland recharge areas the top of the zone of saturation is deep, but such areas generally make poor landfill sites because of the presence of permeable materials that allow downward migration of leachate and lateral migration of landfill gas.

Fine-textured sediments, such as glacial tills, are much more effective than more permeable sands and silts in removing dissolved solids from leachate. Data from the old Du Page County landfill show that dissolved solids in leachate that travels through 5 feet of sandy clay till having a permeability of 10^{-7} centimeters per second are reduced approximately the same amount as solids in leachate that travels through 600 feet of outwash sand and silt having a permeability of approximately 10^{-3} centimeters per second.

At each of the sites studied, ground-water flow patterns are relatively simple, and the hydrogeologic factors responsible for these patterns can, in most cases, be readily inferred. Although the water-quality determinations varied considerably, the over-all distribution of the dissolved solids in the vicinity of these various landfills was, in general, in accord with what would have been predicted if the system of ground-water flow had been known.

The results of this investigation and other studies indicate that environmental problems associated with solid waste disposal are not nearly as serious as those posed by liquid waste disposal operations and pollution of the air through gaseous waste disposal. The technology is available to handle all of the problems associated with solid waste that are likely to occur, with relatively little expense and inconvenience. The major problem appears to be that of implementing this technology and regulating and supervising current and future disposal operations.
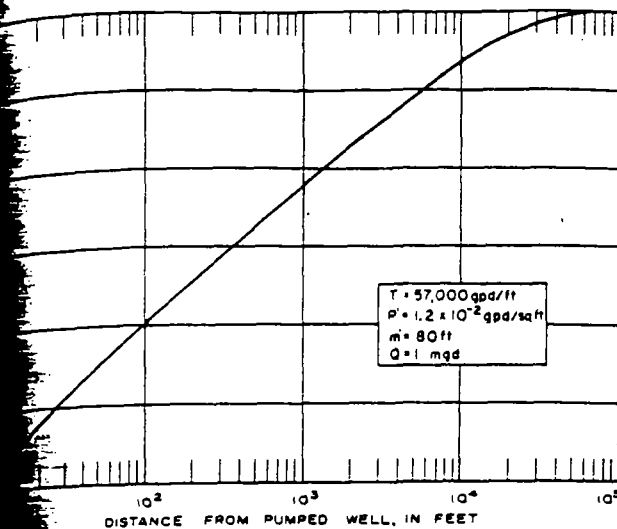
REFERENCES

American Society of Civil Engineers, 1959, Sanitary landfill: Am. Soc. Civil Engrs., Manuals of Eng. Practice, no. 39, 61 p.

Butler, W. J., 1965, A study of the movement of chemical wastes into a ground-water reservoir: Northwestern Univ. [Evanston, Ill.] M.S. thesis, 66 p.

Hughes, G. M., R. A. Landon, and R. N. Farvolden, 1969, Hydrogeologic data from four landfills in northeastern Illinois: Illinois Geological Survey Environmental Geology Note 26, 42 p.

Sawinski, R. J., 1966, Ground-water quality variation at a refuse landfill: South Dakota State Univ. [Brookings, S. D.] M.S. thesis, 89 p.

U. S. Department of Health, Education and Welfare, 1962, Drinking water standards: U. S. Public Health Service Pub. 956, 61 p.

Ground-Water Development in Several Areas of Northeastern Illinois

by T. A. Prickett, L. R. Hoover,

W. H. Baker, and R. T. Sasman

$T = 57,000$ gpd/ft
$P' = 1.2 \times 10^{-2}$ gpd/sq ft
$m' = 80$ ft
$Q = 1$ mgd

DISTANCE FROM PUMPED WELL, IN FEET

61. **Theoretical distance-drawdown graph for lower aquifer at Woodstock**

the lower aquifer. The water-level decline in well
from 1921 to September 1961 was computed using
mathematical model and estimated pumpage data.
puted decline was then compared with the actual
During the month of September 1961, well
as pumped infrequently and the drawdown ob-
the well was mostly due to the effects of pumpage
production wells in the old municipal well
ground-water withdrawals from the old municipal
during September 1961 averaged 1.65 mgd. The
water-level decline in well 5.7d2 was 21.00 feet; a
decline of 22.90 feet was computed with the
model. The computed water-level decline
percent of the actual drawdown.
mathematical model is based on a particular com-
aquifer properties and dimensions. There are
other mathematical models involving several
different combinations of parameters which would
aquifer conditions. It is recognized that the
method of analysis described above provides only
answers on a bulk basis. However, the close
between computed and actual declines indicates
model aquifer and mathematical model closely
geohydrologic conditions of the lower aquifer
stock area. It is reasonable to assume that
quifer and mathematical model may be used
with reasonable accuracy the practical sustained
existing wells in the lower aquifer.
data concerning the hydraulic properties of
and middle aquifers are not sufficient to permit

preparation of a model aquifer and mathematical model
for those aquifers.

Practical Sustained Yield of Existing Well Fields

The model aquifer and mathematical model were used
to determine the maximum amount of water that can be
continuously withdrawn from existing wells screened in
the lower aquifer at Woodstock without eventually lowering
water levels to critical stages below tops of screens, or
exceeding recharge. Computations indicate that the prac-
tical sustained yield of wells in the old municipal well field
is about 2.4 mgd or about 0.9 mgd more than the average
annual rate of pumpage in 1961.

Computations based on the mathematical model for the
lower aquifer and available well-production data indicate
that the practical sustained yield of multi-aquifer wells in
the new municipal well field is about 3 mgd. Of the 3 mgd,
$\frac{1}{4}$ mgd is derived from the upper aquifer, $\frac{3}{4}$ mgd is de-
rived from the middle aquifer, and 2 mgd is derived from
the lower aquifer. Interference between old and new
municipal well fields, effects of partial penetration of pro-
duction wells, and well losses in production wells were
considered in computations.

The pumping rate schedule used in computing practical
sustained yields is given in the following table.

Well number	Average daily pumping rate (mgd)
MCH 44N7E-5.7d1	0.60
5.7d2	0.60
5.7d3	0.60
5.7d4	0.60
MCH 45N7E-32.3c1	1.25
32.4c1	1.00
32.3e1	0.75

The practical sustained yield can be developed by use
of other pumping rate schedules such as pumping wells
5.7d3 and 5.7d4 at rates of about 1.2 mgd each and dis-
continuing use of wells 5.7d1 and 5.7d2 in the old munici-
pal well field. However, an even distribution of with-
drawals from wells in the old municipal well field is more
desirable.

In order to increase the amount of recharge to the
middle and lower aquifers from the 1962 rate of 1.86
mgd to the practical sustained yield of 5.4 mgd, the product
$\Delta h A_e$ must increase to a value 2.9 times the value of
$\Delta h A_e$ in 1962. Thus, full development of the practical
sustained yield will be accompanied by large increases in
the area of diversion and vertical head loss.

LIBERTYVILLE AREA

municipal use at Libertyville and Mundelein
locally from wells in deeply buried dolomite and
gravel aquifers. Since 1905 the average daily
for the two municipal water supplies steadily

increased from 50,000 gallons to 2.14 million gallons in
1962. Continual increases in pumpage caused water levels
to decline about 85 feet at Libertyville and about 60 feet
at Mundelein. Water levels in dolomite wells are not yet

39

at critical stages at Libertyville; pumping levels in sand and gravel wells at Mundelein, however, were below tops of screens in 1962. Available data indicate that the dolomite aquifer is capable of yielding more water than is being withdrawn at present.

Geography and Climate

The Libertyville area is rectangular in shape and includes about 260 square miles in central Lake County, as shown in figure 62. It is bounded on the east by Lake Michigan
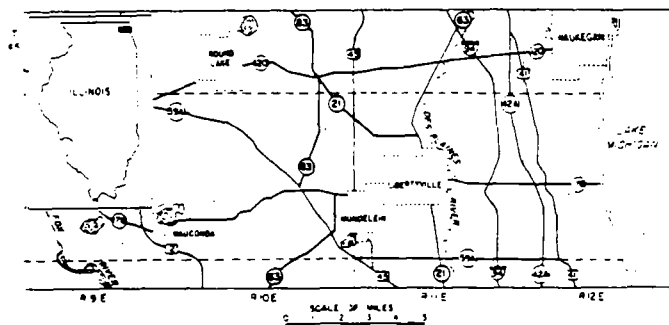
Figure 62. Location of Libertyville area

and on the west by the Fox River, and is between 42°23' and 42°13' north latitude, and between 88°15' and 88°50' west longitude.

Libertyville and Mundelein are in the eastern section of the area (T44N, R11E). State highways 63 and 176 and U.S. 45 pass through the cities as do several railroads.

The Libertyville area lies in the Central Lowland Physiographic Province. The land surface is characterized by hilly topography, broad parallel morainic ridges, lakes, and swamps. Drainage is mainly to the DesPlaines River and Lake Michigan in the eastern part of the area, and to the Fox River and several lakes in the west.

The elevation of the land surface declines from about 900 feet on a ridge 6 miles southwest of Libertyville to about 580 feet along the shore of Lake Michigan, and to about 730 feet in the valley of the Fox River. Maximum relief is about 320 feet.

Graphs of annual and mean monthly precipitation in the Libertyville area given in figures 63 and 64 were compiled from precipitation data collected by the U. S. Weather Bureau at Waukegan (1923-1961). According to these
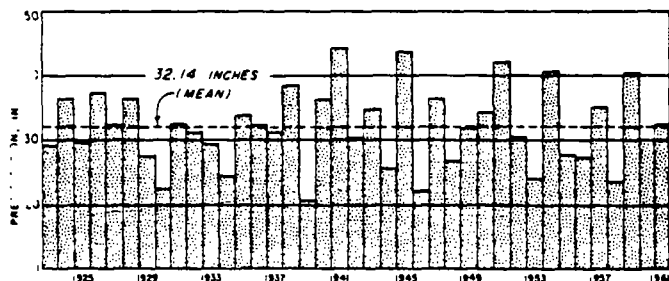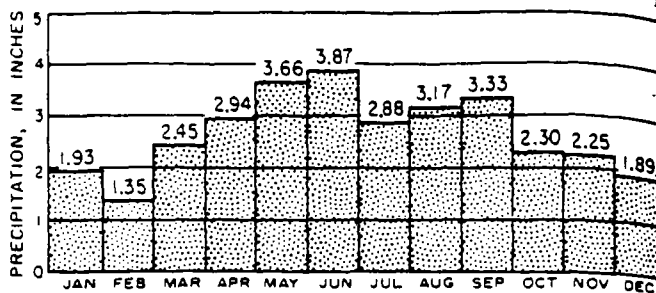
Figure 63. Annual precipitation at Waukegan

Figure 64. Mean monthly precipitation at Waukegan

records the mean annual precipitation is 32.14 inches. On the average, the months of greatest precipitation are May, June, August, and September, each having more than 3.0 inches; February is the month of least precipitation, having less than 1.5 inches.

The annual maximum precipitation amounts occurring on an average of once in 5 and once in 50 years are 36 and 43 inches respectively; annual minimum amounts expected for the same intervals are 27 and 21 inches respectively. Amounts are based on data given in the Atlas of Illinois Resources, Section 1 (1958).

The mean annual snowfall is 31 inches, and the area averages about 46 days with 1 inch or more and about 26 days with 3 inches or more of ground snow cover.

Based on records collected by the U. S. Weather Bureau at Waukegan, the mean annual temperature is 48.7°F. June, July, and August are the hottest months with mean temperatures of 67.3°F, 72.8°F, and 71.6°F respectively; January is the coldest month with a mean temperature of 24.8°F. The mean length of the growing season is 165 days.

Geology

For a detailed discussion of the geology in the Libertyville area the reader is referred to Suter et al. (1959) and Horberg (1950). The following section is based largely upon these two reports.

The Libertyville area is covered mostly with glacial drift which commonly exceeds 200 feet in thickness. The bedrock immediately underlying the glacial drift is mainly dolomite of the Niagaran Series of Silurian age. In the western part of the area the Niagaran Series has been removed by erosion, and dolomite of the Alexandrian Series of Silurian age is the uppermost bedrock. Immediately above the bedrock, the glacial drift contains a thick and fairly extensive deposit of sand and gravel which commonly exceeds 20 feet in thickness. The remainder of the glacial drift is mainly composed of clayey materials (confining bed) and commonly exceeds 175 feet in thickness. Lenses of sand and gravel are intercalated in the confining bed.

A contour map showing the topography of the bedrock surface is shown in figure 65. Features of the bedrock topography were previously discussed by Suter et al. (1959). A bedrock valley extends northeastward across the center
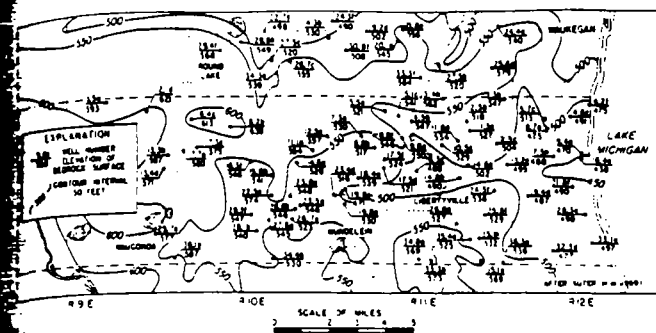
Figure 65. Bedrock topography of Libertyville area

of the Libertyville area. The channel of the bedrock valley exceeds a mile in width in most places, has walls of moderate relief, and averages about 50 feet in depth.

Except in the western part of the Libertyville area, the bedrock surface beneath the glacial drift is formed by rocks of the Niagaran Series as shown in figure 66. The
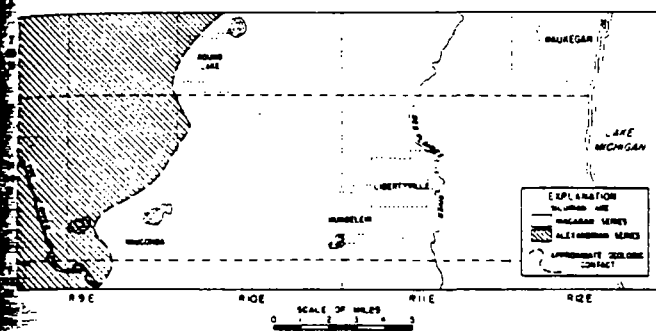

Figure 66. Bedrock geology of Libertyville area

Niagaran Series is composed chiefly of dolomite, although shaly dolomite beds occur at the base. The Niagaran Series in the Libertyville area is relatively more argillaceous than the same series in other parts of northeastern Illinois. The thickness of the Niagaran Series varies, but averages about 60 feet and generally increases from the Niagaran-Alexandrian contact toward the southeastern part of the Libertyville area. The Alexandrian Series is composed chiefly of dolomite; shale and argillaceous dolomite beds occur near the base. The thickness of the Alexandrian Series commonly exceeds 75 feet and averages about 90 feet.

A map showing the thickness of the Silurian rocks is given in figure 67. The thickness of the Silurian rocks in-
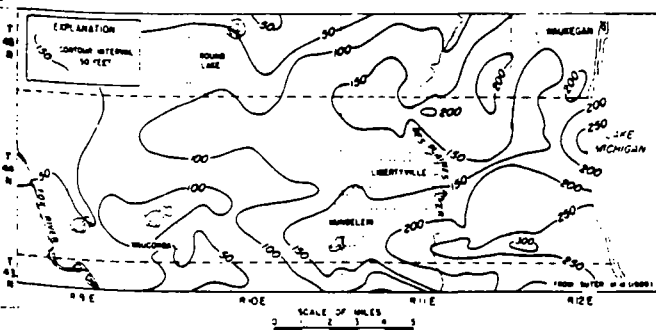

Figure 67. Thickness of Silurian rocks in Libertyville area

creases from less than 50 feet in the western part to over 300 feet in the southeastern corner of the Libertyville area.

The cross section in figure 68 illustrates in general the nature of the unconsolidated deposits above bedrock. The
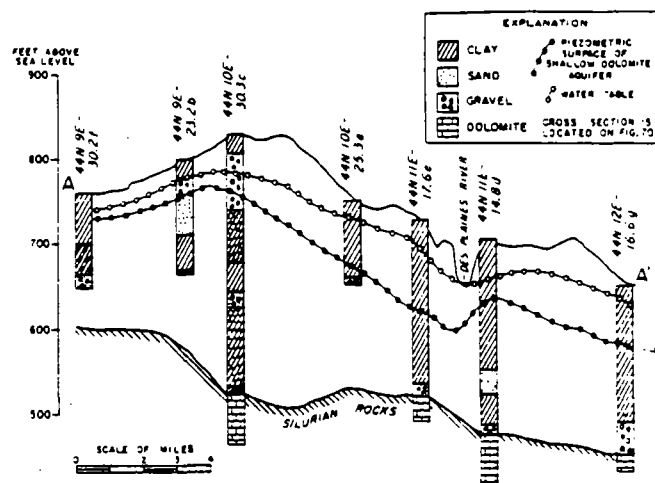

Figure 68. Cross section of glacial drift and piezometric profiles in Libertyville area

unconsolidated deposits are mainly glacial drift, and increase in thickness from less than 100 feet southeast of Libertyville to over 300 feet in the western part of the Libertyville area, as shown on figure 69. The glacial drift
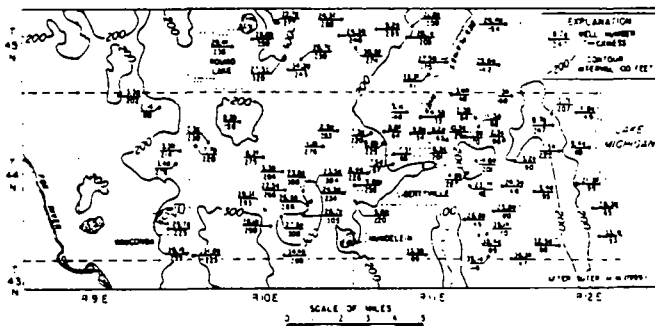

Figure 69. Thickness of unconsolidated deposits overlying bedrock in Libertyville area

consists largely of deposits of till that contain a high percentage of silt and clay.

Logs of wells show that permeable sand and gravel deposits are found in numerous zones within the glacial drift. Sand and gravel occur at the base of the glacial drift over most of the Libertyville area. The thickness of this zone is variable but averages 20 feet except in the vicinity of the channel of the buried bedrock valley near the center of the Libertyville area. Based on logs of a few wells which completely penetrate the glacial drift, the thickness of this basal sand and gravel deposit increases in the vicinity of the bedrock valley and commonly exceeds 40 feet. Geologic data suggest that the materials of the basal sand and gravel deposit may be predominantly fine-grained in the vicinity of the buried bedrock valley, a characteristic that would make development of high capacity wells diffi-

41

Table 16. Logs of Selected Wells and Test Holes in Libertyville Area

Well number	Formation	Thickness (ft)	Depth (ft)	Well number	Formation	Thickness (ft)	Depth (ft)
LKE—				14.8d	clay	150	150
43N11E—					quicksand	30	180
9.3f	clay	60	60		clay	36	216
	sand and gravel, some clay	20	80		gravel	11	227
	clay	16	96		rock	—	285
	sand and gravel	11	107	17.6e	clay	200	200
	clay	12	119		gravel	14	214
	rock	—	119		rock	—	235
				24.5f	clay	35	35
44N9E—					sand	20	55
14.4h	clay	116	116		mud and gravel	10	65
	clay and boulders	10	126		sand	10	75
	clay	74	200		mud and gravel	20	95
	sand and gravel	15	310		mud	15	110
23.2b	clay	22	22		sand, mud and gravel	20	130
	gravel	18	40		gravel	12	142
	gray sand	48	88		rock	—	—
	clay	41	129	25.8f	clay	155	155
	sand and gravel	5	134		gravel	3	158
36.4d	drift	200	200		rock	—	177
	gravel	27	227	26.8d	clay	100	100
	rock	—	227		gravel	45	145
					rock	—	165
44N10E—				32.7d	clay	185	185
8.2g	clay	57	57		gravel and clay	5	190
	sand	18	75		sand and gravel	2	192
	clay	60	135	35.4h	clay	120	120
	gravel	21	156		sand	65	185
	rock	—	262		rock	—	1600
17.4f	clay	23	23				
	gravel	20	43	44N12E—			
	clay	91	134	7.4f	clay	140	140
	gravel	10	144		gritty sand	45	185
25.3e	clay	90	90		gravel	2	187
	sand and gravel	7	97	16.6g	clay	160	160
30.3c	clay	26	26		gravel	44	204
	gravel	64	90		rock	—	204
	quicksand	62	152				
	clay	28	180	45N10E—			
	sand and gravel	18	198	26.3c	clay	226	226
	clay and sand	90	288		fine sand	3	229
	sand and gravel	10	298		rock	—	233
	bedrock	—	358	30.1f	clay	160	160
					fine sand	50	210
44N11E—					clay	10	220
4.7f	clay	80	80		gravel	5	225
	sand	5	85				
	clay	30	115	45N11E—			
	gravel	16	131	21.8a	clay	140	140
	rock	—	143		clay and sand	50	190
9.8e	clay	60	60		gravel	10	200
	sand	30	90		rock	—	204
	clay	60	150	26.2e	clay	74	74
	gravel	4	154		sandy clay	14	88
	rock	—	178		clay	61	149
10.8h	clay	54	54		clay and gravel	22	171
	sand	18	72		sand and gravel	4	175
	hardpan	6	78		rock	—	177
	sand	23	101	30.6e	clay	180	180
	hardpan	27	128		quicksand	70	250
	gravel	2	130		gravel	15	265
	rock	—	141		rock	—	285
11.8d	clay	130	130	MCH—			
	sand and gravel	40	170	44N9E—			
	rock	—	200	30.2f	clay	60	60
					clay and gravel	35	95
					gravel and sand	17	112

ult or impossible. Additional subsurface information is needed to determine the thickness and character of the glacial drift especially in the vicinity of the buried bedrock valley.

Relatively impermeable deposits (confining bed) consisting of sandy and silty clay and gravel overlies the basal sand and gravel deposits. The thickness of these clayey materials varies considerably but averages about 175 feet. Deposits of permeable sand and gravel of limited areal extent are interbedded in the confining bed. Many wells in the Libertyville area penetrate these interbedded sand and gravel aquifers and supply moderate quantities of water. Logs of wells and other geologic data suggest that interbedded sand and gravel aquifers are found at most places; however, extensive test drilling is needed in order to locate deposits large enough in areal extent to support heavy pumpage.

Drillers logs of selected wells and test holes for which geologic data are available are given in table 16. The locations of the wells and test holes are shown in figure 70.
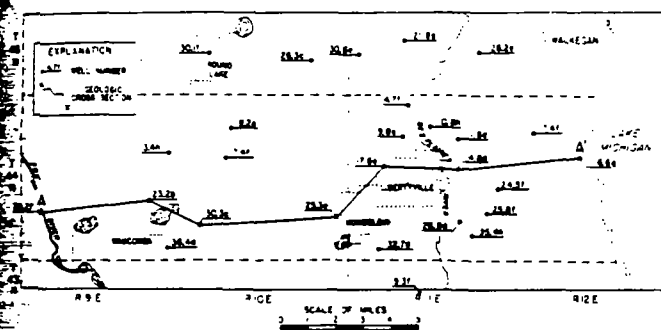

Figure 70. Location of selected wells and test holes in Libertyville area

Occurrence of Ground Water

Ground water in the Silurian dolomite aquifer occurs in joints, fissures, fractures, solution cavities, and other openings. The water-yielding openings are irregularly distributed, both vertically and horizontally. The conformable piezometric surface and available geohydrologic data suggest that the dolomite aquifer is permeated by numerous openings which extend for considerable distances and are interconnected on an areal basis. The weathered zone of the upper part of the dolomite aquifer has a relatively high permeability; the Niagaran Series is generally more permeable than the Alexandrian Series. Suter et al. (1959) state that among the many factors that may be responsible for the inconsistent productivity of the Silurian dolomite aquifer are:

1. Differences in development of solution zones with respect to bedrock topography.
2. Differences in depth of ground-water circulation. For example, shallow impermeable shales in a given dolomite area may limit solution downcutting and promote extensive enlargement of channels in the soluble rock above.

3. Differences in permeability of the overlying drift.
4. Differences in solubility of the various dolomite units.

Leaky artesian conditions exist where till or other fine-grained deposits overlie the Silurian dolomite aquifer and impede or retard the vertical movement of ground water, thus confining the water in the Silurian dolomite aquifer under artesian pressure. Under leaky artesian conditions, water levels in wells rise above the top of the Silurian dolomite aquifer to stages within the fine-grained deposits.

Ground water in the glacial drift is obtained mainly from sand and gravel aquifers underlying or interbedded with glacial till. Because of their irregularity of occurrence, glacial drift aquifers are often more difficult to locate than bedrock aquifers. The difficulties are compensated for in part by lower costs of drilling and pumping, often by water that is cooler or of better quality, and at some places by greater yields. The ground water in the sand and gravel aquifers in the Libertyville area occurs under leaky artesian conditions.

Water-Yielding Properties of Aquifers

Silurian Dolomite Aquifer

During the period 1929-1961, well-production tests were made by water well contractors and the State Water Survey on more than 80 dolomite wells in and near the Libertyville area. The well-production tests consisted of pumping a well at a constant rate and frequently measuring the drawdown in the production well. Drawdowns were measured usually with an airline or electric dropline; rates of pumping were measured by means of a circular orifice at the end of the pump-discharge pipe.

The results of the tests are summarized in table 17. The lengths of tests range from 1 to 24 hours and average 8 hours. Pumping rates range from 10 to 740 gpm. Diameters of inner casings range from 4.5 to 12 inches and the average radius of inner casings averages about 1/3 foot.
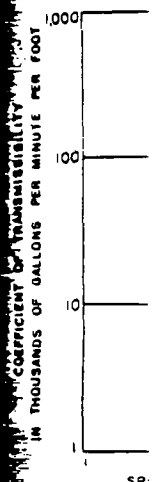
Unfortunately, very few step-drawdown tests were made in the Libertyville area. Values of well loss were estimated for all wells based on the results of studies made on 40 step tests on dolomite wells in northeastern Illinois (Csallany and Walton, 1963). Well losses were subtracted from observed drawdowns, and specific capacities adjusted for well losses were computed.

Several values of t and r_w, and a coefficient of storage of 0.0003, were substituted into the nonequilibrium equation to determine the relationships between specific capacity and the coefficient of transmissibility for various values of r_w^2/t, and results are shown in figure 71. This graph, specific capacities adjusted for well losses, and data concerning the lengths of tests and radii of wells in table 17 were used to estimate theoretical coefficients of transmissibility of the Silurian dolomite aquifer in the vicinities of the

43

Table 17. Specific-Capacity Data for Dolomite Wells in Libertyville Area

Owner	Depth (ft)	Diam. (in)	Penetration (ft)	Year drilled	Year of test	Length of test (hr)	Non-pumping level (ft)	Pumping rate (gpm)	Draw-down (ft)	Unadjusted specific capacity (gpm/ft)	Unadjusted specific capacity per foot of penetration (gpm/ft²)	Adjusted specific capacity (gpm/ft)	Adjusted specific capacity per foot of penetration (gpm/ft²)	Estimated coefficient of transmissibility (gpd/ft)
E. Sayewski	247	4.5	100	1934	1934	5	75	16	20	0.81	0.008	0.83	0.008	1,500
G. Greswell	252	6	40	1933	1933	24	50	30	80	0.38	0.009	0.41	0.010	900
I. Bates	207	6	39	—	—	—	38	50	7	7.15	0.183	7.60	0.195	13,000
Valenti	310	10	71	1958	1958	7	112	740	3	247.00	3.480	426.00	6.000	1,000,000
Elm Construction Co.	310	10	71	1958	1958	3	119	716	2	358.00	5.180	568.00	8.000	1,300,000
M. McDoo	247	6	15	1937	1937	4	82	40	3	13.35	0.890	13.95	0.931	24,000
Pure Oil, Barrington	305	10	85	1945	1945	12	84	663	9.5	70.00	0.825	137.00	1.620	310,000
Albani Real Est.	330	6	43	1956	1958	8	135	240	35	6.85	0.159	9.95	0.232	13,000
Wm. Rueffer	274	4.5	10	1941	1941	8	90	16	13	1.23	0.125	1.27	0.127	21,000
Mt. St. Joseph (V)	400	10	112	1949	1949	8	106	113	67	1.69	0.015	2.12	0.019	2,000
C. E. Johnson	278	6	11	1941	1941	24	101	40	9	4.45	0.404	4.72	0.428	8,700
Lake Zurich (V)	443	6	143	1949	1949	1	40	135.5	3	13.31	0.093	13.92	0.098	23,000
Lake Zurich (V)	421	10	149	1951	1951	1.5	108	396	7	56.60	0.380	82.00	0.550	150,000
G. Reed	197	6	30	1939	1939	2	45	10	20	0.50	0.017	0.51	0.017	700
C. H. Parson	192	6	17	1940	1941	14	33	40	15	2.67	0.057	2.86	0.170	5,000
L. Schaufler	211	6	17	1937	1937	4	46	40	14	2.86	0.163	3.06	0.180	5,500
R. L. Huszaah	202	6	9	1934	1944	—	8	35	14	2.50	0.278	2.65	0.295	4,100
Kopp Farm	350	6	81	1958	1958	5	110	100	2	50.00	0.617	55.50	0.685	120,000
Towner Sbd.	280	6	39	1957	1957	—	85	50	75	0.67	0.017	0.75	0.019	1,100
Vernon Hills, Inc.	190	6	17	1961	1961	24	74	50	38	1.32	0.078	1.46	0.086	3,200
J. D. Allen	162	6	44	1941	1941	16	38	15	7	2.14	0.049	2.20	0.050	4,100
Wm. Johnson	350	—	230	1938	1957	24	53	236	44	5.36	0.023	7.94	0.035	13,000
Chevy Chase CCb.	280	—	160	1938	1958	24	53	236	44	5.37	0.034	8.33	0.052	14,000
W. Wecker	228	6	18	1935	1935	8	68	25	32	0.78	0.043	0.82	0.045	1,400
H. E. LeRoy	275	6	81	1940	1940	10	30	75	10	7.50	0.093	8.27	0.012	16,000
Ill. Toll Hwy. Comm.	330	6	115	1958	1958	21	63	30	3	10.00	0.087	10.23	0.090	20,000
Island Lake (V)	182	4.5	20	1940	1940	8	50	20	50	0.40	0.020	0.42	0.021	900
Wauconda (V)	231	8	13	1939	1939	8	39	210	20	10.50	0.807	13.85	1.065	27,000
Wauconda (V)	257	12	31	1939	1939	8	36	318	44	7.23	0.233	11.20	0.362	20,000
Wauconda (V)	325	12	32	1957	1957	8	25	287	155	1.85	0.058	4.00	0.125	5,500
Mundelein (V)	270	6	37	1954	1954	—	64	183	13	14.05	0.380	17.45	0.474	32,000
A. T. McIntosh & Co.	264	12	31	—	—	—	80	350	143	2.45	0.079	5.80	0.187	10,000
Loch Lomond Sbd.	358	8	91	1953	1953	—	71	60	10	6.00	0.066	6.70	0.074	11,000
A. T. McIntosh & Co.	270	6	39	1954	1954	—	64	183	13.5	13.60	0.348	13.90	0.357	24,000
M. J. Boyle	358	12	60	1944	1944	8	—	330	20	16.50	0.276	25.50	0.425	47,000
Wm. M. Paris	351	6	5	1942	1942	8	135	25	2	12.50	2.500	12.90	2.580	24,000
Leesley Nursery	252	8	81	—	1929	9	20	60	19	3.16	0.039	3.48	0.043	6,000
Leesley Nursery	255	8	90	1956	1959	—	40	100	70	1.43	0.016	1.75	0.019	2,600
E. J. Burns (Sbd.)	168	12	46	1954	1954	—	14	115	75	1.53	0.033	1.80	0.039	2,600
B. Cooper	242	6	79	1959	1959	24	47	125	93	1.35	0.017	1.75	0.022	
E. P. Doerr	260	6	50	1959	1959	8	77	226	36	6.27	0.125	8.89	0.177	17,000
Casey	342	6	104	1941	1941	4	60	35	80	0.44	0.004	0.49	0.005	1,000
Farm	301	6	74	1940	1940	4	67	30	28	1.07	0.014	1.14	0.015	2,000
Libertyville (V)	297	6	99	1955	1958	8	53	200	24	8.34	0.086	11.00	0.131	25,000
Libertyville (V)	300	6	102	1955	1955	6	33	630	152	4.15	0.041	5.25	0.052	8,700
Libertyville (V)	251	—	64	1929	1958	—	96	300	57	5.26	0.082	8.92	0.140	15,000
Libertyville (V)	320	8	120	1961	1961	8	95	247	40	6.20	0.052	8.00	0.067	15,000
Libertyville (V)	240	8	40	1921	—	5	—	70	35	2.00	0.050	2.28	0.057	3,500
Libertyville (V)	287	12	115	1947	1947	3	15	330	37	8.90	0.078	14.90	0.130	24,000
Libertyville (V)	286	6	129	1946	1958	—	20	150	14	10.70	0.083	12.85	0.100	20,000
Libertyville (V)	227	12	71	1951	1951	1.8	39	305	13	23.60	0.332	33.30	0.470	55,000
Mundelein (V)	285	12	46	1930	1930	—	64	120	57	2.11	0.046	2.64	0.057	4,100
Mundelein (V)	213	10	1	1946	1946	6	90	125	60	2.08	2.080	2.63	2.630	4,000
I. Florsheim	178	10	9	1935	1935	24	37	100	27	3.71	0.412	4.35	0.483	8,000
E. J & E. RR	215	5	25	1959	1961	3	73	55	34	1.62	0.065	1.80	0.072	3,000
Cuneo, Inc.	231	6	51	1954	1954	—	50	50	140	0.36	0.007	0.46	0.009	600
Cuneo, Inc.	231	6	51	1954	1954	—	50	130	128	1.02	0.020	1.38	0.027	2,000
A. A. Gilchrist	222	4.5	17	1941	1941	4	50	10	4	2.50	0.147	2.54	0.150	4,500
Abbott Lab	270	8	113	1952	1952	—	44	30	88	0.34	0.003	0.37	0.003	500
Great Lakes Hosp.	200	—	20	—	—	9	25	110	35	3.15	0.158	3.78	0.189	6,000
T. H. Donnelly	201	8	12	1936	1936	10	40	55	55	1.00	0.083	1.12	0.093	3,500
C. Olmstead	295	8	125	1937	1937	3	55	25	120	0.21	—	0.23	0.002	400
Fox Lane Hills Sbd.	383	—	111	1954	1954	—	60	30	140	0.21	0.002	0.24	0.002	300
Wooster Lake Co.	200	—	8	—	1932	1	30	70	2	35.00	4.370	37.20	4.650	70,000
E. Ross	265	6	65	1947	1947	—	40	40	10	4.00	0.062	4.20	0.065	6,800
A. Hallman	235	4.5	9	1939	1939	6	45	10	10	1.00	0.112	1.02	0.114	1,700
L. Hennier	263	4	15	1951	1951	—	60	10	5	2.00	0.199	7.20	0.248	12,000
Shorewood Ridge Water Co.	314	6	81	1952	1952	—	45	200	81	2.47	0.031	3.66	0.045	6,000
L. B. Harris	278	6	40	1952	1952	—	45	128	20	6.40	0.160	7.70	0.193	13,000
Shorewood Ridge Water Co.	342	12	54	1947	1948	23	41	100	158	0.63	0.012	0.79	0.015	1,300
Public Service Co. of N. Ill.	267	—	57	1953	1953	—	35	25	27	0.93	0.016	1.11	0.020	1,200
Round Lake Park	313	10	53	1944	1944	8	46	100	74	1.35	0.025	1.67	0.032	2,800
Grays Lake (V)	337	12	87	1958	1958	8	80	442	145	3.05	0.035	9.82	0.113	17,000
Boysen Water Co.	279	6	29	1939	1939	6	46	150	26	5.78	0.199	7.20	0.248	15,000
Boysen Water Co.	313	10	53	1944	1944	8	46	100	54	1.85	0.035	2.25	0.043	5,000
Round Lake (V)	350	6	120	1914	1945	1.5	40	175	10	17.50	0.146	21.00	0.175	40,000
Round Lake (V)	359	10	133	1945	1945	7.5	51	288	107	2.66	0.020	5.10	0.058	10,000
Waukegan C'tryside Sbd.	285	6	75	1957	1957	—	25	22	175	0.13	0.002	0.14	0.002	200
Island Lake (V)	190	4.5	20	1941	1941	8	70	10	40	0.25	0.012	0.26	0.013	500

test sites. Specific capacities adjusted for well losses were then further adjusted to a common radius and pumping period, based on estimated coefficients of transmissibility and the graphs in figure 71. The average radius (4 inches) and pumping period (8 hours) based on data in table 17 were used as the bases (see base line in figure 71).
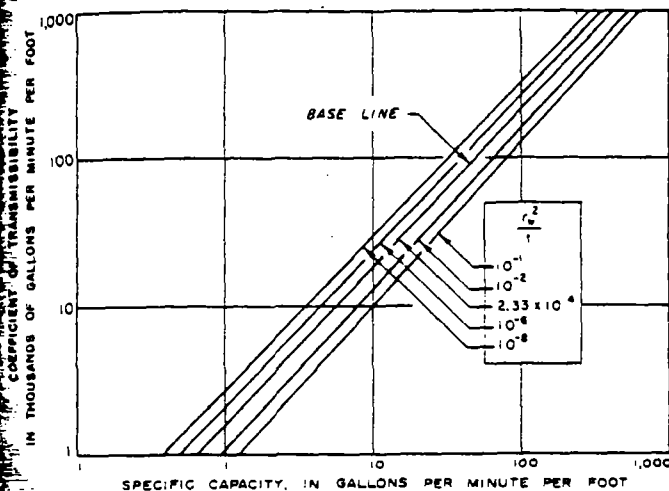

Figure 71. Coefficient of transmissibility versus specific capacity for several values of well radius and pumping period

No great accuracy is inferred for the adjusted specific capacities or computed coefficients of transmissibility because they are based on estimated well-loss constants; however, they come much closer to describing the relative yields of wells than do the observed specific capacities based on pumping rates, pumping periods, and radii which vary from well to well. Based on the average adjusted specific capacity in table 17, the coefficient of transmissibility of the Silurian dolomite aquifer averages about 10,000 gpd/ft in the Libertyville area.

In general, the specific capacity of a dolomite well increases with the depth of penetration; the upper part of the Silurian dolomite aquifer is usually the most productive, however. The total depths of penetration of wells into dolomite were determined from well logs and sample studies of drill cuttings, and are given in table 17. Adjusted specific capacities were divided by the total depths of penetration to obtain the adjusted specific capacities per foot of penetration in table 17.

Wells were divided into two categories, those which penetrate less than 33 percent of the Silurian age rocks and those which penetrate more than 33 percent of the Silurian age rocks. Adjusted specific capacities per foot of penetration for wells in the two categories were tabulated in order of magnitude, and frequencies were computed by the Kimball (1946) method. Values of specific capacity per foot of penetration were then plotted against percent of wells on logarithmic probability paper as shown in figure 72.
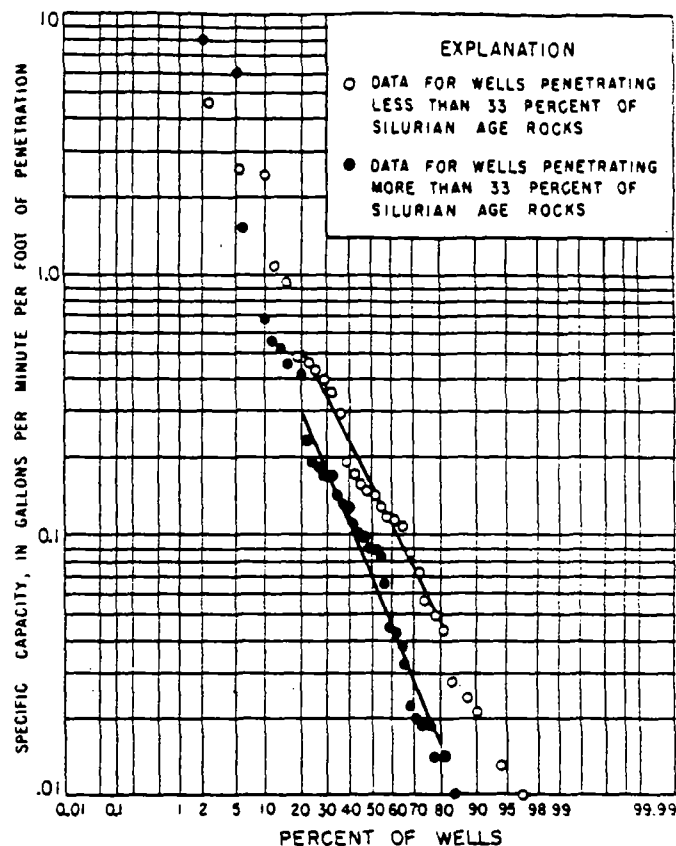

Figure 72. Specific-capacity frequency graphs for dolomite wells in Libertyville area

Specific capacities per foot of penetration decrease as depths of wells increase, indicating that the upper part of the Silurian dolomite aquifer is more productive than the lower part.

Glacial Drift Aquifers

Numerous rural and residential water supplies, but only a few municipal, commercial, and industrial supplies, are obtained from glacial drift aquifers. The largest development of sand and gravel aquifers is at Mundelein, and the village has one sand and gravel well that is capable of producing 750 gpm. Data for municipal and industrial wells obtaining water from glacial drift aquifers in the Libertyville area are given in table 18. This tabulation indicates that the specific capacity of sand and gravel wells ranges from 1.0 to 47.4 gpm/ft and averages about 14 gpm/ft. The average depth and diameter of wells are 140 feet and 10 inches, respectively, and the average thickness of the aquifer at well sites is 40 feet. Of the 33 wells listed only two were not equipped with screens. The average length of screen used in the 31 screened wells is 15 feet. Specific-capacity data in table 18 indicate that the coefficient of transmissibility of the sand and gravel aquifers in the Libertyville area ranges between 2000 and 90,000 gpd/ft and averages 25,000 gpd/ft.

45

Table 18. Specific-Capacity Data for Wells in Glacial Drift Aquifers in Libertyville Area

well number	Owner	Depth (ft)	Diam. (in)	Screen length (ft)	Screen diam. (in)	Thickness of aquifer (ft)	Date of test	Non-pumping level (ft)	Pumping rate (gpm)	Draw-down (ft)	Specific capacity (gpm/ft)
LKE—											
4--79E—											
.3f	Jewel Tea Co.	163	30	30	—	—	1953	78	520	14	37.1
..6e	Kendall Co.	112	12	15	12	14	1962	81	343	16.5	20.8
43N10E—											
nn.6h	Lake Zurich (V)	214	10	no screen	—		1921	95	100	20	5.0
.1a	Twin Orchard CCb.	136.5	10	15	10	17	1958	18	644	50	12.9
4..711E—											
15.2c	Do-Mor Day Camp	55	6	30	6	33	1961	9	44	8	5.5
19.8h	G. W. Traer	190	12	17	12	90	1948	42	127	13	9.8
4 79E—											
.8b	Island Lake Water Co.	116	10	24	10	—	1946	29	425	11	38.5
21.7f	Island Lake Water Co.	95	8	11	8	—	1960	16	360	21	17.1
44N10E—											
4.3g	Mundelein (V)	140	12	24	12	30	1954	69	800	21	38.1
4.1g	Mundelein (V)	165	20	20	20	15	1959	104	1000	44	23.0
44N11E—											
10.5c	Green Valley Bldrs.	36	5	4	5	12	1958	19	80	9.5	8.4
1.5d	Oak Grove Schl.	99	6	—	—	26	1957	45	25	6	4.2
5.1a	Foulds Milling	202	8	14	8	190	1945	7	275	84.5	3.3
19.3b	Mundelein (V)	106	12	20	20	—	1955	56	500	19	26.3
21.7f	Libertyville (V)	83	—	30	—	30	1935	28	380	14	27.1
n2.4d	M. Dall	34	6	6	6	15	1941	7	75	8	9.4
0.6c1	Mundelein (V)	200	6	8	6	—	1949	47	103	45	2.3
0.6c2	Mundelein (V)	194	8	20	8	24	1951	50	138	105	1.3
45N9E—											
1.2h	Fox Lake Hills Sbd.	130	8	15	8	15	1954	16	290	76	3.8
.4a	Fox Lake Hills Sbd.	126	10	10	10	—	1954	38	600	30	20.0
.1g	Fox Lake (V)	135	16	16	15	45	1941	36	284	4	75.8
15.5c1	Hilldale Manor Sbd.	123	6	15	6	—	1954	67	253	20	12.7
15.5c2	Hilldale Manor Sbd.	123	8	16.75	8	—	1954	72	281	17	16.5
N10E—											
2.7h	Lindenhurst (V)	165	8	16	8	16	1961	52	300	36	8.4
17.7h	Round Lk. Beach (V)	174	12	20	12	21	1948	49	500	52	9.6
18.2f	Round Lk. Beach (V)	215	8	6	8	16	1947	50	75	75	1.0
21.5h	Shorewood Sbd.	253	4.5	no screen		73	1942	30	10	10	1.0
5N11E—											
7.6b	Hoag Farm	145	12	25	12	110	1949	31.5	289	77	3.8
31.5h	Wildwood Sbd.	145	6	12	6	12	1950	95	53	23	1.9
31.4h	Wildwood Sbd.	173	6	14	6	—	1952	108	201	6	33.5
33.8h	B. K. Evans	151	8	4	6	5	1940	65	100	10	10.0
ICH—											
44N9E—											
20.2f	Island Lake Water Co.	122	8	10	8	107	1954	24	75	50	1.5

Probable Yields of Dolomite Wells

Because the productivity of the Silurian dolomite aquifer is inconsistent, it is impossible to predict with a high degree of accuracy the yield of a well before drilling at any location. Probable range of yields of wells can be estimated from the frequency graphs in figure 72 and data on the thickness of the Silurian dolomite aquifer. Probable specific capacities of wells in figure 73 were estimated as the product of the specific capacity per foot of penetration measured in 50 percent of the existing wells (see figure 72) and aquifer thickness (see figure 67). Specific capacities equal to or less than 10 gpm/ft can be expected in large areas in the western part of the Libertyville area where the thickness of the Silurian dolomite aquifer is less than 150 feet. Specific capacities equal to or less than 15 gpm/ft can be expected in areas in the eastern part of the Liberty-ville area where the thickness of the dolomite of Silurian age commonly exceeds 150 feet.

Probable specific capacities were in turn multiplied by available drawdowns based on water-level data (see figure
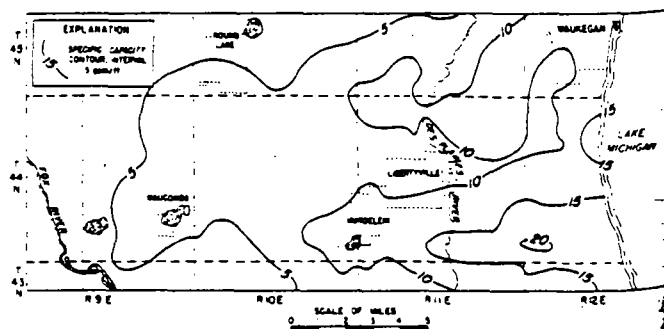

Figure 73. Estimated specific capacities of wells in Silurian dolomite aquifer in Libertyville area

85) to estimate the probable yields of wells. Nonpumping levels were limited to the top of the Silurian dolomite aquifer.

The probable range of yields of dolomite wells is shown in figure 74. It is possible to drill what is essentially a dry
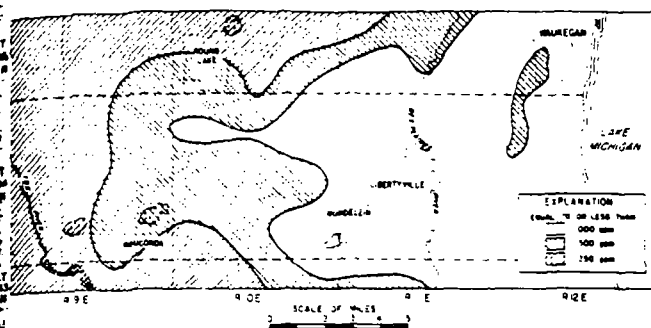

Figure 74. Estimated yields of wells in Silurian dolomite aquifer in Libertyville area

hole at any location. Based on data for 50 percent of existing wells, however, the chances of obtaining a well with a yield of 500 gpm or more are good except in the western part of the Libertyville area where the Silurian rocks are thin. Thus, the yield of the Silurian dolomite aquifer is probably high enough to support heavy industrial or municipal well development in all but a small part of the Libertyville area.

For design purposes, the reader may wish to base the computation of the probable yield of a well on a specific capacity with a particular frequency other than 50 percent. In this event the probable yield indicated in figure 74 is multiplied by the ratio of the specific capacity with the selected frequency (see figure 72) and the specific capacity with a 50 percent frequency.

Construction Features of Wells and Pumps

The production wells located at Libertyville and Mundelein serve as examples of the usual type of well installations
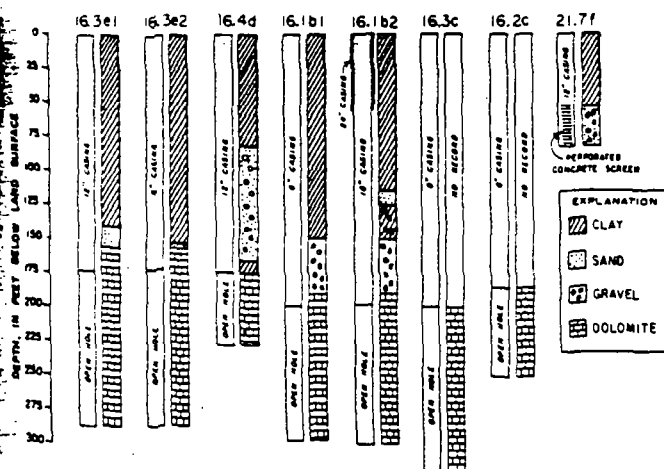
Figure 75. Generalized construction features and logs of production wells at Libertyville

Figure 76. Location of production wells at Libertyville

found in the Libertyville area and are described in detail below. The construction features of the eight existing production wells at Libertyville are illustrated in figure 75; locations of the wells are shown in figure 76. Dolomite
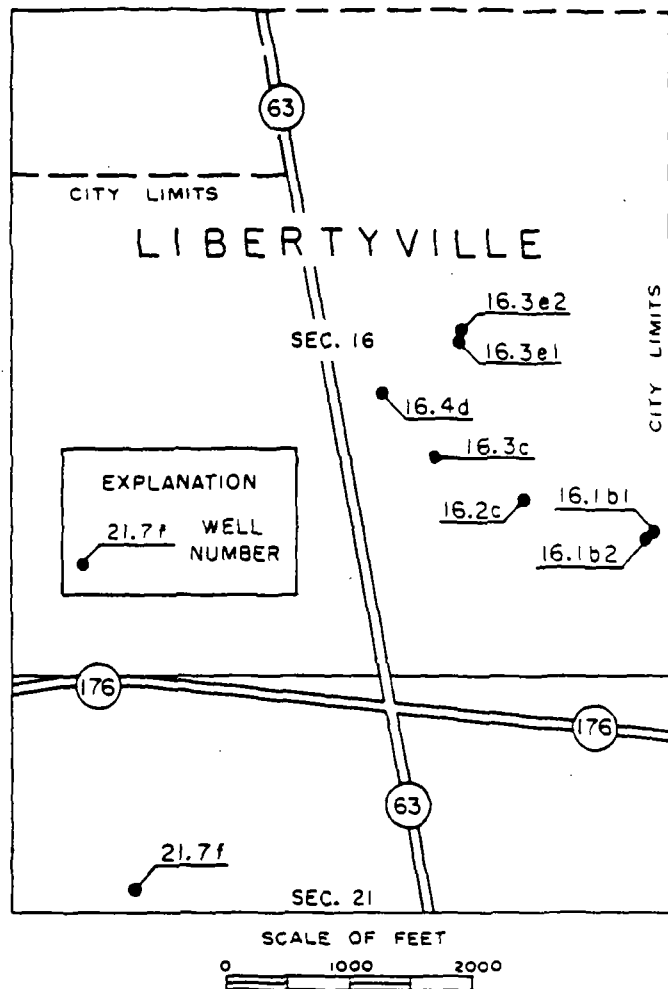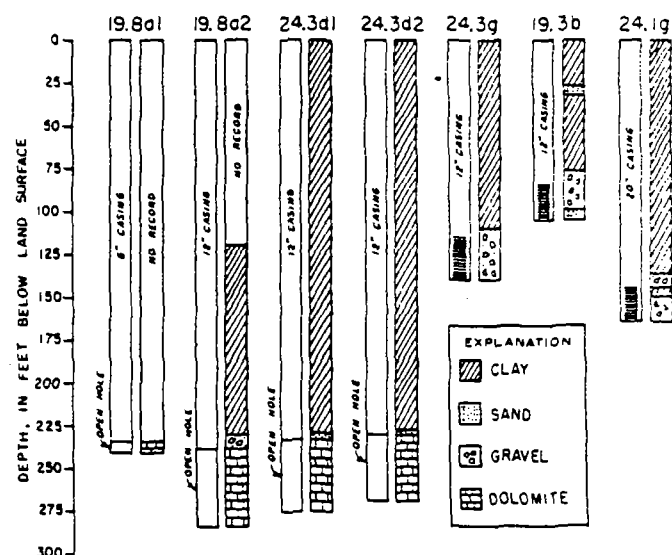

Figure 77. Generalized construction features and logs of production wells at Mundelein

vells range in depth from 227 feet to 320 feet and range .n diameter from 6 inches to 16 inches. The dolomite wells at Libertyville penetrate almost the entire thickness of Silurian rocks.

The construction features of the Mundelein wells are shown in figure 77; the locations of these production wells are shown in figure 78. The four wells penetrating the
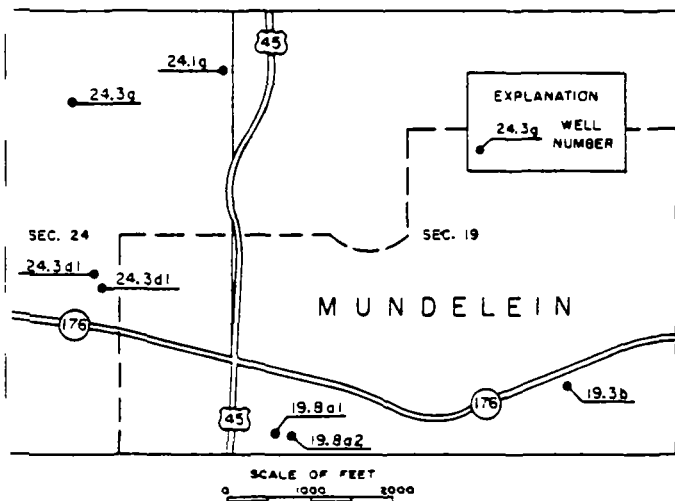
Figure 78. Location of production wells at Mundelein

Silurian dolomite aquifer at Mundelein range in depth from 241 to 285 feet and penetrate only the upper part of the Silurian rocks. The three sand and gravel wells are natural pack wells and range in depth from 106 to 165 feet. No data are available on the slot size of screen openings.

Pumps in wells in the Silurian dolomite aquifer at Libertyville and Mundelein are powered by 7.5 to 75 horsepower electric motors. The number of bowl stages ranges from 4 to 25. Column pipes have lengths ranging from 100 to 260 feet and diameters ranging from 4 to 8 inches. Details on pump installations at Libertyville and Mundelein are given in table 19.

Table 19. Description of Pumps in Wells at Libertyville and Mundelein

Well number	Pump rating capacity/head (gpm)/(ft)	Number of stages	Column pipe length (ft)	Column pipe diam. (in)	Motor horsepower
LKE—					
44N10E—					
24.1g	700/468	9	141	10	125
24.3d1	280/350	11	220	6	40
24.3d2	100/217	22	170	4	15
24.3g	700/250	5	111	8	50
44N11E—					
16.1b1	200/380	4	220	4	30
16.1b2	700/290	6	260	8	75
16.2c	300/230	8	200	6	25
16.3c	240/139	—	—	—	25
16.3e1	325/200	12	100	6	20
16.3e2	150/241	23	130	4	15
16.4d	500/225	6	140	8	40
19.3b	500/217	6	80	8	40
19.8a1	70/270	25	150	4	7.5
19.8a2	190/280	12	175	8	30
21.7f	400/190	9	73	7	30

Ground-Water Withdrawals

Distribution of pumpage in 1962 from dolomite and glacial drift aquifers within the Libertyville area is shown in figure 79. Data in table 20 indicate that of the total
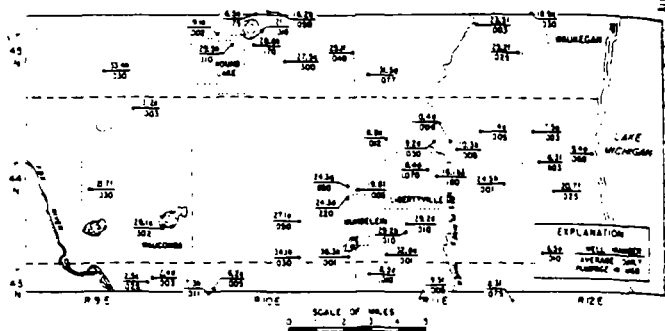
Figure 79. Distribution of pumpage from shallow aquifers in Libertyville area, 1962

water pumped from wells in 1962, 63 percent was derived from the Silurian dolomite aquifer and 37 percent from glacial drift aquifers. In 1962 withdrawals for public water-supply systems amounted to 41 percent of the total pumpage; industrial pumpage was 12 percent of the total; and domestic pumpage was about 47 percent of the total.

Table 20. Distribution of Pumpage from Wells in Libertyville Area, Subdivided by Source and Use, 1962

Use	Pumpage from glacial drift aquifers (mgd)	Pumpage from Silurian dolomite aquifer (mgd)	Total pumpage (mgd)
Public	1.069	2.580	3.649
Industrial	.829	.203	1.032
Domestic	1.410	2.830	4.240
Total	3.308	5.613	8.921

Public use data are classified in this report according to three main categories: 1) *public,* including municipal, subdivisions, and institutional; 2) *industrial,* including commercial, industrial, golf courses, irrigation, and cemeteries; and 3) *domestic,* including rural farm and rural nonfarm.

Most water-supply systems furnish water for several types of use. For example, a public supply commonly includes water used for drinking and other domestic uses, manufacturing processes, and lawn sprinkling. Industrial supplies may also be used in part for drinking and other domestic uses. No attempt has been made to determine the final use of water within categories. Any water pumped by a municipality is called a public supply, regardless of its use.

The reliability of pumpage data varies greatly. Municipal pumpage is nearly always metered in cities and large villages, but many small villages and subdivisions operate without meters. Only a few of the larger industries meter their supplies. Pumpage data for municipalities and some of the larger industries are systematically recorded. Pumpage from farm wells and individual residential wells is estimated on the basis of detailed use surveys. For these
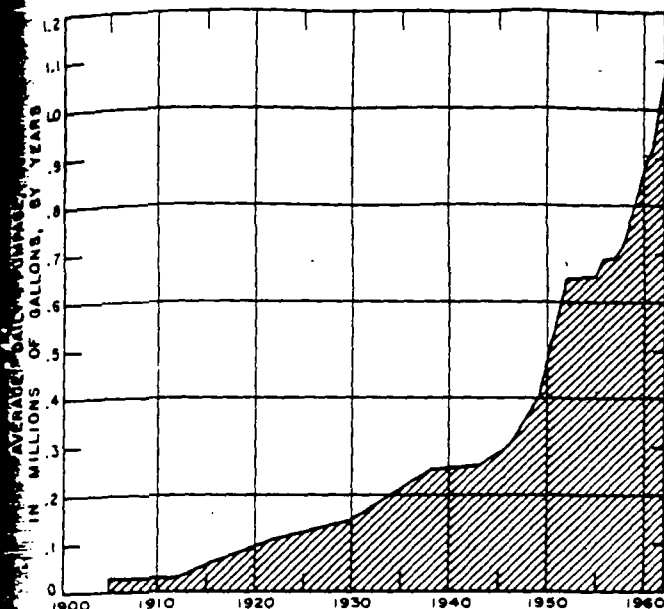
Figure 80. Estimated ground-water pumpage at Libertyville, 1905-1962

reasons it is often difficult to estimate pumpage precisely. The village limits of Libertyville and Mundelein and immediately surrounding areas constitute the areas of greatest ground-water withdrawals in the Libertyville area. Pumpage data show that the municipal use of water was about the same (1 mgd) in Libertyville and Mundelein in 1962.

Pumpage at Libertyville has grown at an accelerating rate since the original installation of a water supply system in 1905. The pumpage in 1905 was about 30,000 gpd and increased to about 290,000 gpd in 1945 at an annual rate
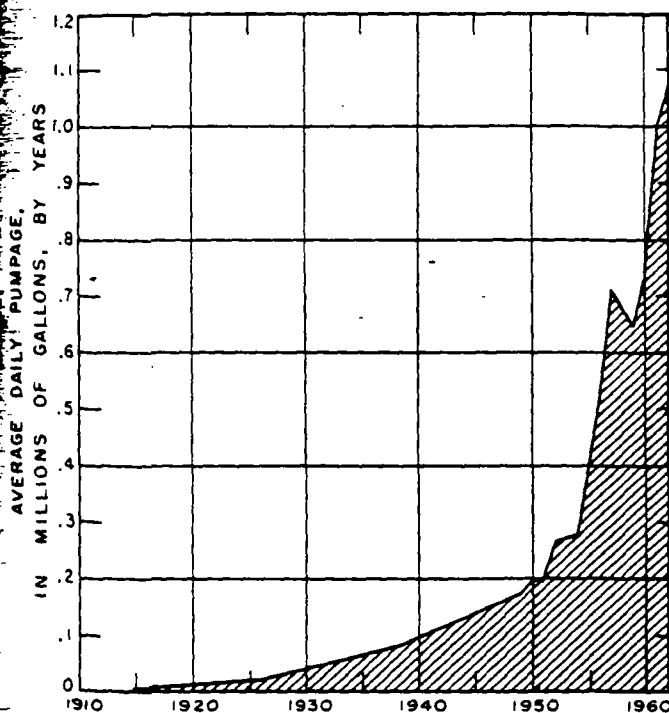
of increase of 6500 gpd/yr. This rate increased to 47,00 gpd/yr after 1945 and in 1962 the daily pumpage amounted to 1,070,000 gpd (see figure 80).

Prior to 1936 the majority of the water pumped a Libertyville was from glacial drift aquifers. After 1936 when sand and gravel wells were abandoned, the municipal water supply was obtained from dolomite wells.

Mundelein installed a water supply in 1915 when the demand was almost 20,000 gpd. Pumpage increased to about 200,000 gpd in 1950, having grown at a rate of 5000 gpd/yr. After 1950 pumpage greatly accelerated, reaching 1,070,000 gpd by 1962 (see figure 81). During the period 1950 to 1962 the annual increase in pumpage exceeded 72,000 gpd/yr. This rapid increase may in part be explained by the addition of the Loch Lomond Subdivision to the municipal water supply system; Mundelein purchased the wells and distribution system of the subdivision in 1956.

Prior to 1954, all water supplied to Mundelein consumers was obtained from wells penetrating the Silurian dolomite aquifer. During the period 1954-1957, pumpage from sand and gravel wells gradually increased in glacial drift aquifers as shown in figure 82. In 1957, 66 percent of the water was obtained from glacial drift aquifers and 34 percent from the Silurian dolomite aquifer. In 1962, 20 percent or 220,000 gpd was derived from the Silurian dolomite
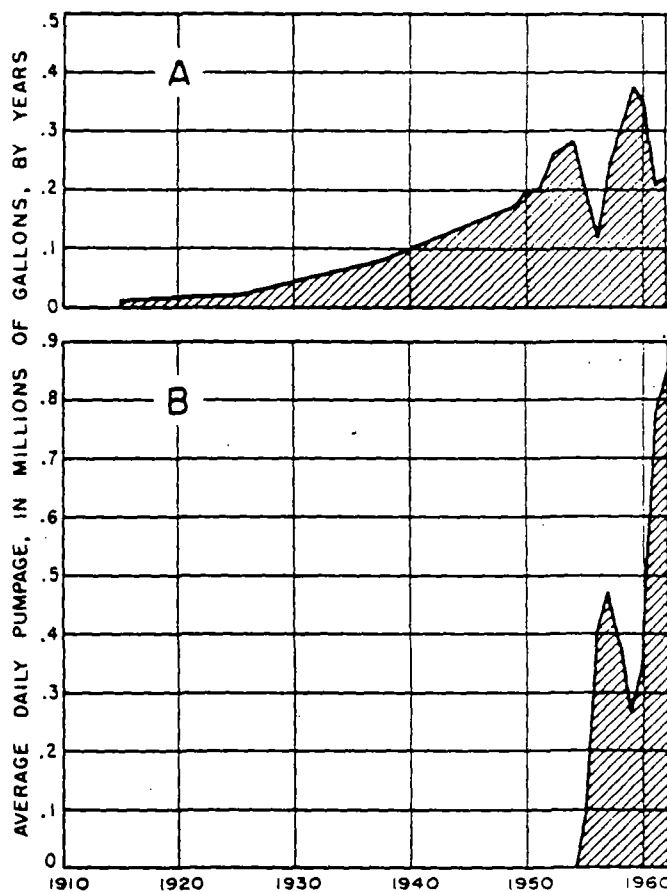

Figure 81. Total ground-water pumpage from aquifers at Mundelein, 1915-1962

Figure 82. Ground-water pumpage from Silurian dolomite aquifer (A) and from sand and gravel aquifers (B) at Mundelein, 1915-1961

49

aquifer and 80 percent or 850,000 gpd from glacial drift aquifers.

Leakage through Maquoketa Formation

n many instances pumpage is not the only withdrawal fi..m the dolomite aquifer. In the Libertyville area a vertical hydraulic gradient prevails which allows the down-w.d movement of water through the Maquoketa Forma-ti i. The Maquoketa Formation of Ordovician age is largely shale and is the confining bed between shallow a--ifers and the heavily pumped Cambrian-Ordovician A .ifer in northeastern Illinois. In 1962 the piezometric surface of the Cambrian-Ordovician Aquifer was, on the average, 200 feet below the water table in the Libertyville a a, and downward movement of water through the .\.quoketa Formation was appreciable under the influence of large differentials in head between shallow deposits and tl Cambrian-Ordovician Aquifer. The quantity of leak-a through the Maquoketa Formation can be computed from the following form of Darcy's law:

$$Q_c = (P'/m') \triangle h A_c \qquad (13)$$

w :re:

Q_c = leakage through Maquoketa Formation, in gpd

P' = vertical permeability of Maquoketa Formation, in gpd/sq ft

m' = saturated thickness of Maquoketa Formation, in ft

A_c = area of Maquoketa Formation through which leakage occurs, in sq ft

$\triangle h$ = difference between the head in the Cambrian-Ordovician Aquifer and the head in shallow de-posits, in ft

P ed on data given by Walton (1960), the average vertical p meability of the Maquoketa Formation in the Liberty-ville area is estimated to be about 0.00005 gpd/sq ft. The area of Maquoketa Formation through which leakage o urs is about 260 square miles. The average $\triangle h$ was d..ermined to be about 200 feet; the average thickness of the Maquoketa Formation is about 200 feet. Substitution o these data into equation 13 indicates that the leakage t ough the Maquoketa Formation within the Liberty-ville area was about 0.37 mgd in 1962.

Fluctuations of Water Levels and Their Significance

'Vater-level measurements were made infrequently in s eral wells in the Libertyville area between 1917 and 1962. Changes in water levels shown in figure 83 are in-dicative of conditions in general at Libertyville. It should b emphasized that water levels in figure 83 are nonpump-ing levels.

The average elevation of the piezometric surface at L ertyville in 1917 was probably about 675 feet, and flow-
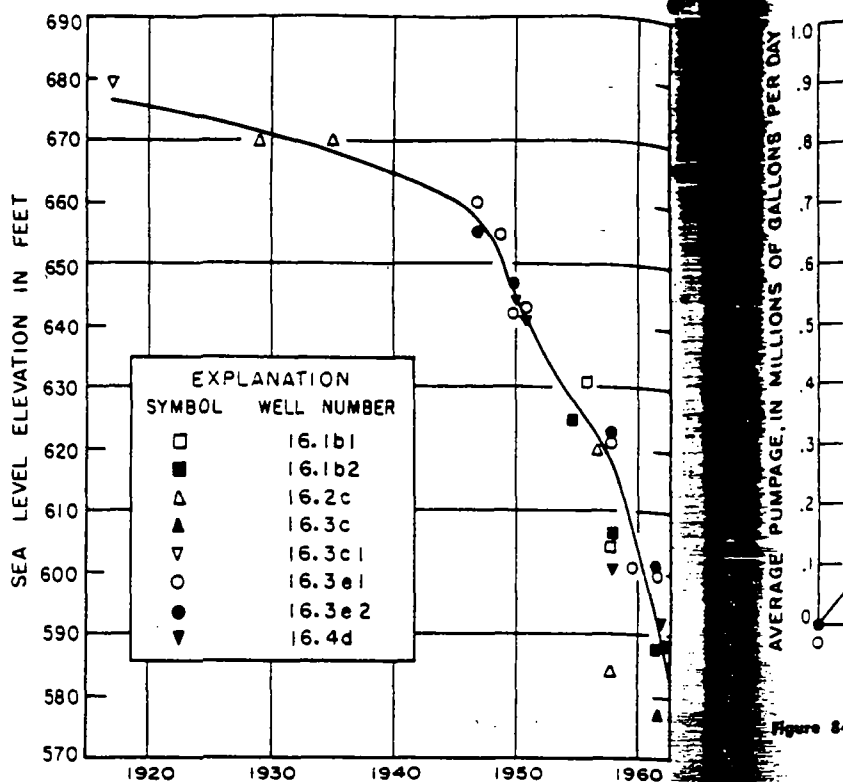

Figure 83. Water levels in wells at Libertyville, 1917-1962

ing wells existed in the vicinity. By 1950 water levels had declined in response to continual withdrawals of water to an average elevation of 645 feet. Thus, in a period of 33 years, water levels declined 30 feet or at an average rate of about 0.9 foot per year. As the result of continual in-creases in pumpage, water levels declined from an average elevation of 645 feet in 1950 to an average elevation of 590 feet in 1962. The average rate and total decline of water levels, 1950 through 1962, were 4.6 feet per year and 55 feet, respectively.

A comparison of the water-level hydrograph shown in figure 83 and the pumpage graph shown in figure 80 indicates that in general water-level decline has been proportional to the rate of pumpage. Average water-level declines in wells plotted against corresponding average rates of pumpage at Libertyville are shown in figure 84. The consistent relationship between decline and pumpage is apparent. Approximately 12,000 gpd were obtained with each foot of decline. The consistent relationship between decline and pumpage indicates that in the past recharge has balanced discharge. If pumpage is kept constant, water levels decline at a decreasing rate with time and eventually stabilize at a stage lower than that measured prior to pumping. Water levels do not stabilize but continue to decline if pumpage constantly increases. Pumpage in the past at Libertyville has not remained constant but has increased almost without interruption, as shown in figure 80; as a result, water levels have never stabilized but have declined continuously throughout the period of development.
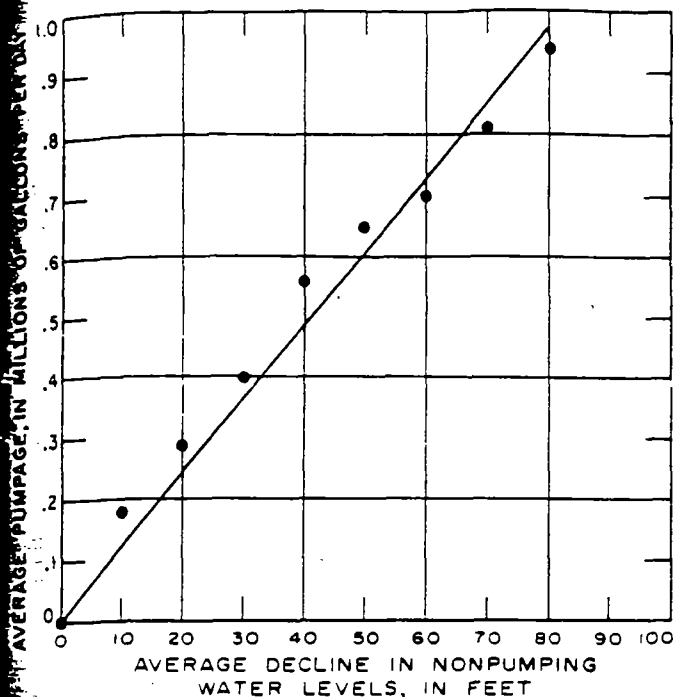
Figure 84. Relation between average pumping rate and decline of water levels at Libertyville

Nonpumping levels in wells at Mundelein were measured with less frequency than wells at Libertyville; available water-level data are summarized in table 21. Water-level trends cannot be determined from the short-term records. Table 22 lists pumping levels in wells at Libertyville and Mundelein. Pumping-level measurements were made at infrequent intervals. Pumping levels in sand and gravel wells 19.3b and 24.1g at Mundelein were below tops of

Table 21. Nonpumping Levels in Wells at Mundelein

Well number	Date measured	Depth to water (ft below measuring point)
LKE—		
44N10E—		
24.1g	1959	104
24.1g	1960	114
24.1g	1961	115
24.1g	1962	120
24.3d1	1954	82
24.3d1	1960	95
24.3d1	1961	98
24.3d1	1962	100
24.3d2	1954	91
24.3d2	1961	79
24.3d2	1962	89
24.3g	1954	69
24.3g	1959	90
24.3g	1960	92
24.3g	1961	87
24.3g	1962	95
44N11E		
19.3b	1955	56
19.3b	1961	70
19.3b	1962	75
19.6a	1946	90
19.8a1	1915	40
19.8a1	1926	40
19.8a1	1961	110
19.8a2	1930	64

Table 22. Pumping Levels in Wells at Libertyville and Mundelein

Well number	Date measured	Pumping rate (gpm)	Depth to water (ft below measuring point)
LKE—			
44N10E—			
24.1g	1962	750	148
24.3d1	1961	320	175
24.3d2	1962	110	107
24.3g	1962	460	109
44N11E—			
16.1b1	1955	728	170
16.1b2	1955	630	188
16.3c	1961	247	135
16.3e1	1962	300	95
16.3e2	1962	150	95
16.4d	1958	500	105
19.3b	1962	200	92

screens in 1962; pumping levels in dolomite wells at Libertyville averaged about 60 feet above the top of the dolomite aquifer in 1962.

Configuration of Piezometric Surface of Aquifers

In order to determine areas of recharge and discharge and directions of ground-water movement in the Silurian dolomite aquifer, a piezometric surface map was made (figure 85). Data on nonpumping levels in table 23 were used to prepare the map.
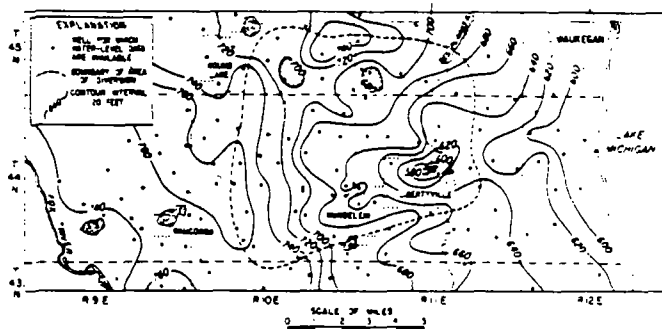
Figure 85. Piezometric surface of Silurian dolomite aquifer in Libertyville area, July–August 1962

The piezometric surface map in figure 85 represents the elevation to which water will rise in a well completed in the Silurian dolomite aquifer, and does not usually coincide with the position of the water table in shallow sand and gravel aquifers. The map was prepared from water-level measurements made mostly during the months of July and August 1962. The majority of observation wells shown on the map are open in the Silurian dolomite aquifer; however, some wells are open to deeply buried sand and gravel deposits above the Silurian dolomite aquifer. The Silurian dolomite aquifer and deeply buried sand and gravel aquifers have slightly different water levels at the same location (for example, see wells LKE 44N10E-23.8e and LKE 44N10E-23.8f in table 23). The piezometric surface could not be mapped using only the available water-level data for the Silurian dolomite aquifer; water-

51

Table 23. Water-Level Data for Wells in Silurian Dolomite Aquifer and in Deeply Buried Sand and Gravel Aquifers in Libertyville Area

Well number	Owner	Depth (ft)	Aquifer	Depth to water (ft below measuring point)	Land surface elevation (ft above MSL)	Water level elevation (ft above MSL)	Date measured
LKE—							
43N9E—							
1.3a	R. A. Gillis	265	s&g	94.7	856	761	8/14/62
2.1b	A. R. Rein	166	s&g	24.8	790	765	7/5/62
3.6d	C. Schleifer	200	dol	30.7	765	734	7/30/62
43N10E—							
1.8d	Rogers	—	s&g	70.7	775	704	8/13/62
2.6e	O. Ohenauf	320	s&g	37.5	781	743	8/10/62
3.3g	D. J. Hodges	300	—	82.4	825	743	7/10/62
4.3a	R. J. Kreuser	—	s&g	140.7	885	744	7/10/62
6.3f	R. Rieke	300	s&g	109.8	865	755	7/24/62
8.3g	E. Mahachek	—	—	123	873	750	7/5/62
24.1b	Twin Orchard CCb.	136	s&g	9	740	731	3/29/62
43N11E—							
3.1h	G. Brown	139	dol	22.8	686	663	8/16/62
6.5e	Towner Sbd.	180	dol	39	728	689	4/26/62
6.8g	Diamond Lake Schl.	234	s&g	69.18	760	691	2/8/62
8.3f	Vernon Hills Sbd.	150	s&g	30	715	685	4/26/62
43N12E—							
4.2e	Barat College	71	dol	67.6	695	627	8/20/62
6.5a	Fisher	—	dol	35.1	677	642	8/17/62
16.2h	Old Elm Golf Club	184	dol	27.14	650	623	8/17/62
44N9E—							
1.2d	J. Hoffman	212	s&g	56.44	820	764	6/21/62
3.2f	B. Schmidt	200	s&g	25.7	788	762	6/26/62
9.2h	E. L. Fisher	—	s&g	28.6	784	755	6/27/62
10.3c	W. Chandler	170	s&g	21	776	755	6/26/62
11.3e	Natural gas pipeline	280	s&g	27.2	792	765	6/25/62
12.2b2	W. Roney	160	s&g	42.3	805	763	7/17/62
13.6b	D. Lexow	150	s&g	42.7	810	767	7/13/62
14.2h	K. Sorlie	150	s&g	49.8	812	762	7/18/62
16.1c	—	84	s&g	17.9	774	756	7/3/62
21.8a	J. Conigli	131	s&g	22.5	764	742	7/27/62
22.5c	J. Dowell	80	s&g	27.8	790	756	7/26/62
23.7f	R. Sanca	208	s&g	42	797	755	6/27/62
24.2e	M. Arendt	255	dol	75.4	839	764	7/31/62
24.3e	B. Jennings	190	s&g	28	795	767	7/12/62
25.1f	A. Christensen	142	s&g	44.0	800	756	7/31/62
26.7b	A. Daumke	220	s&g	23.5	772	742	7/26/62
27.2e	R. Norcross	160	s&g	35.8	782	746	7/27/62
28.2f	J. Filippo	178	s&g	13.3	749	736	6/29/62
34.1g	P. Piccolo	275	dol	13.8	750	736	7/3/62
35.1f	W. H. Bode	165	s&g	42.8	787	744	7/24/62
36.2e	J. Baska	230	s&g	19.2	789	770	7/6/62
44N10E—							
1.4c	Suhling	265	s&g	107.0	810	703	10/16/61
2.5d	G. Titus	335	dol	116.1	812	696	8/8/62
3.5f	F. Krene	254	dol	103.2	823	720	3/26/62
4.6h	V. Campbell	—	s&g	43.92	785	741	3/26/62
6.1a	A. Hartle	200	s&g	55.2	809	754	7/18/62
7.5d	R. Hartle	—	s&g	65.9	821	765	7/19/62
8.3g	Schroeder Nursery	200	s&g	41.85	792	750	6/21/62
9.2e	L. Behm	256	dol	75.53	811	735	10/9/61
10.4d	—	—	s&g	101.4	814	713	8/9/62
11.3f	G. Stode,	—	dol	145.18	822	677	10/16/61
12.5e	R. Meyer	265	dol	138.46	815	677	9/18/61
13.6c	V. Kahn	165	s&g	128.98	802	673	9/18/61
14.7h	H. Grabbe	205	s&g	174.67	853	678	9/26/61
15.7e	A. M. Wirtz	314	dol	99.90	825	725	10/9/61
16.4g	L. Itta	329	dol	73.9	812	738	7/18/62
17.2e	R. Kebro	151	s&g	41.2	802	761	7/18/62
18.2e	J. H. Betjmann	285	dol	40.1	800	760	7/13/62
19.1g	J. Epstein	—	s&g	62.6	825	762	8/1/62
20.4g	R. Bielewitz	200	s&g	52.3	813	761	7/12/62
21.4g	Fremont Schl.	300	dol	60.3	810	750	7/11/62
23.8e	B. Small	298	s&g	142.31	850	708	10/19/61
23.8f	M. Behm	304	dol	145.55	848	702	9/29/61

52

Table 23 (Continued)

Date measured	Well number	Owner	Depth (ft)	Aquifer	Depth to water (ft below measuring point)	Land surface elevation (ft above MSL)	Water level elevation (ft above MSL)	Date measured
	LKE— 44N10E-(Cont'd)							
3/14/62	24.3d1	Mundelein (V)	276	dol	98	750	652	5/22/61
7/5/62	24.3d2	Mundelein (V)	270	dol	89	750	661	1/27/62
62	24.3g	Mundelein (V)	140	s&g	103	750	647	1/26/62
	24.1g	Mundelein (V)	165	s&g	119	777	658	1/31/62
1/13/62	24.7d	E. Kingman	111	s&g	103.67	793	689	9/3/61
1/10/62	26.7f	A. S. Hanson	312	dol	142.53	825	682	3/26/62
1 62	27.2g	W. A. Singer	180	s&g	128.13	845	717	10/9/61
1 62	28.8g	A. Dahlquist	300	dol	48.5	810	762	7/20/62
1/2 62	29.3b	R. V. Jones	—	s&g	88.5	850	762	7/23/62
7/5/62	30.6e	C. Lochmoon	375	dol	53.0	810	757	7/23/62
1/20 62	31.6a	A. Niemic	285	s&g	98.8	850	751	7/24/62
	32.1g	A. Mioriello	280	s&g	109.2	870	761	7/11/62
1/10/62	33.6e	E. M. Olsen	125	s&g	117.0	875	758	7/19/62
1/26/62	34.1b	Schwerman	300	s&g	63.2	808	745	8/14/62
2/9 62	35.7b	N. B. Heath	300	s&g	49.9	796	746	8/10/62
1/2 62	36.7h	G. Reimey	—	s&g	76.0	767	691	8/8/62
	44N11E-							
1/20/62	1.3h	T. E. Wilson	185	dol	69.29	720	651	4/2/62
1/17/62	2.3e	R. E. Anlliser	85	s&g	57.2	705	648	5/21/62
1 62	5.3d	—	—	s&g	20.14	705	685	4/16/62
	6.6h	Anderson	250	s&g	94.61	785	690	10/16/61
1/21/62	7.6e	G. Christienson	204	s&g	71.6	750	678	4/16/62
1/26/62	8.8e	Leesley Nursery	255	dol	34.13	710	676	4/16/62
1/2 62	9.2d	North Libertyville	168	dol	37.95	662	624	7/13/62
1/2 62	9.1h	J. V. Casey	150	s&g	39.41	672	633	4/16/62
1/25/62	9.7c	Cities Serv. Sta.	90	s&g	42.79	712	669	4/2/62
1/17/62	11.5g	Ascension Cmty.	160	s&g	68.29	695	627	3/29/62
1 52	13.2d	C. Vennett	178	s&g	98.00	715	617	4/19/62
1 62	14.1f	Atkinson Farm	110	s&g	78.00	695	617	4/19/62
7/3 62	14.8d	E. Harrison	200	s&g	61.70	696	634	4/20/62
1/27/62	15.5c	T. McFayden	180	s&g	23.58	648	624	4/20/62
1/26/52	16.1b1	Grocery Store Prod.	100	s&g	72.0	660	588	6/19/62
1/2 52	16.1b2	Grocery Store Prod.	90	s&g	63.0	658	595	6/19/62
1/3 52	16.3c	Libertyville (V)	320	dol	108	685	577	4/5/62
1/12/62	16.3e2	Libertyville (V)	286	dol	74	675	601	4/5/62
1/31/62	16.4d	Libertyville (V)	227	dol	98	690	592	4/5/62
1/2 52	16.1b1	Libertyville (V)	297	dol	70	638	588	4/5/62
1/2 52	16.1b2	Libertyville (V)	300	dol	70	658	588	4/5/62
1/25/62	16.3e1	Libertyville (V)	287	dol	74	675	601	4/5/62
7/3/62	17.7e	Quaker Oats Res. Fm.	180	dol	95.91	730	634	4/23/62
1/24/52	19.3b	Mundelein (V)	106	s&g	75	743	668	5/22/62
7/1 52	19.8a1	Mundelein (V)	242	dol	110	765	655	5/22/61
	19.8a2	Mundelein (V)	285	dol	35	765	730	5/22/61
	19.6a	Mundelein (V)	213	dol	90	765	675	5/22/61
16/61	19.8f1	St. Mary's of the Lake Sem.	300	dol	162	729	567	—
3/8/62	19.8f3	St. Mary's of the Lake Sem.	295	dol	77	740	663	2/5/62
1/2 52	22.5e	C. Shen	—	—	6.26	654	648	4/26/62
1/2 52	23.5f	R. L. Vachherm	65	s&g	54	705	651	11/5/61
1/18/62	25.8b	H. R. Vahnke	—	dol	43.40	692	649	4/26/62
1/19/62	26.8b	—	126	s&g	35.17	674	639	4/28/62
1/21 2	27.4b	—	—	dol	flowing	650	650	4/30/62
1/9 1	28.3g2	—	86	s&g	29.55	709	679	4/30/62
1/5/62	29.7b	R. P. Hillinger	96	s&g	52.48	740	688	4/30/62
16/61	30.6c1	Mundelein (V)	200	s&g	46.5	745	698	6/8/49
18/61	30.6c2	Mundelein (V)	194	s&g	50	780	730	3/1/51
18 1	31.6b	P. Baldino	75	s&g	30.85	752	721	5/3/62
26 1	32.8c	E.J.&E. RR	215	dol	63.27	728	665	3/19/62
1/9/61	36.8h	—	185	dol	40.5	688	647	8/16/62
18/62	**44N12E-**							
18 2	6.5d	Abbott Lab.	270	dol	68	690	622	8/9/62
13 2	7.7f	Pagoda Motel	162	s&g	85.12	694	599	7/13/62
1/1/62	8.7g	J. Wasniewski	180	s&g	122.4	715	593	8/9/62
12/62	9.4a	Shore Acres	285	dol	73.42	653	580	8/9/62
11/62	9.7d	Shore Acres CCb.	210	s&g	55.5	645	590	8/10/62
19 1	17.5h	H. E. Doney	350	dol	125	720	595	8/10/62
29 1	18.3f	Goodyear Tire & Rubber	144	s&g	50	680	630	1/7/62
	20.7f	Natural Marble Co.	165	dol	58.74	668	609	8/13/62

Table 23 (Continued)

Well number	Owner	Depth (ft)	Aquifer	Depth to water (ft below measuring point)	Land surface elevation (ft above MSL)	Water level elevation (ft above MSL)	Date measured
LKE—							
44N12E-(Cont'd)							
21.8f	Lake Bluff (V)	498	dol	90.80	680	5█	7/20/62
29.7d	C & NW RR	224	dol	41.42	675	6█	8/13/62
30.7a	Le Wa Farm	175	dol	45.53	673	6█	8/13/62
45N9E-							
24.7f	Gavin Schl.	180	dol	67.7	802	734	8/22/62
26.5b	F. O. Mark Trust	200	s&g	25.9	766	740	8/24/62
36.4e	D. Rowden	—	s&g	48.9	802	753	8/21/62
45N10E-							
14.5f	—	200	s&g	50.00	777	727	8/23/62
19.5c	H. Renner	—	s&g	45.8	779	733	8/22/62
20.3g	R. Below	—	dol	43.2	761	718	8/23/62
22.7a	F. Ruszkowski	250	s&g	86.2	800	714	8/3/62
23.7d	G. Halsey	200	s&g	66.5	783	717	8/23/62
26.2b	—	105	s&g	40.15	790	750	6/20/62
27.5c	Grays Lake (V)	337	dol	93	793	700	11/22/61
28.2b	Grays Lake Gelatin	275	dol	91.2	801	710	8/7/62
29.4f	C. Junge	160	dol	83.8	800	716	8/23/62
30.4c	V. A. Taseher	206	dol	42.9	770	727	8/21/62
32.1h	J. Writz	240	dol	49.2	790	741	6/20/62
32.8c2	H. Vanderspool	—	s&g	75.6	812	736	6/21/62
34.5e	W. Hintz	230	dol	109.3	795	686	8/7/62
35.2e	C. Stemler	250	s&g	86.6	793	707	8/1/62
36.1b	H. L. Milk Farm	260	s&g	124.37	811	687	10/16/61
45N11E-							
19.7h	E. Lohuck	225	s&g	75.15	790	715	8/23/62
21.8e	E. Huffhines	109	s&g	49.9	751	701	8/23/62
26.3d	L. Buraadt	175	dol	48	727	679	8/23/62
27.7f	—	140	s&g	48.63	697	648	8/24/62
30.3e	Peterson	360	dol	96.5	790	693	6/19/62
30.8h	J. S. Porto	226	s&g	33	778	745	8/3/62
31.7h	Wildwood Sbd.	145	s&g	130	816	684	11/20/61
31.5g	Wildwood Sbd.	173	s&g	145	810	665	11/20/61
32.4g	L. Bristol	115	s&g	64.7	768	703	6/19/62
33.4d	Serbian Monastery	70	s&g	26	705	679	6/18/62
34.4a2	E. S. Richardson	90	s&g	67.15	713	646	6/18/62
45N12E-							
32.8a	N. Shore Cmty.	168	s&g	101.50	711	609	8/9/62
MCH—							
44N9E-							
5.5g1	C. Fritzsche	198	s&g	15.3	766	751	6/28/62
5.5g2	C. Fritzsche	220	s&g	6.2	756	750	6/28/62
20.7h	A. Shustitsky	130	dol	14.9	755	740	7/27/62
29.6c	E. Kocmoud	140	s&g	63.7	795	731	7/27/62

level data for wells penetrating deeply buried sand and gravel aquifers were used to augment data for wells in the Silurian dolomite aquifer. On the basis of measured water levels in a few closely spaced wells drilled to different depths, it is probable that the piezometric surfaces of the Silurian dolomite aquifer and deeply buried sand and gravel aquifers are in general very similar. Accordingly, it is believed that the contours on figure 85 can be used to determine the approximate directions of movement of ground water, the average hydraulic gradients of the piezometric surface, and the area of diversion of pumping in the Silurian dolomite aquifer.

A pronounced cone of depression is centered around Libertyville and Mundelein. Other cones of depression are present at Grays Lake and at Wildwood Subdivision in the north-central part of the Libertyville area. Ground-water movement is in all directions toward well fields or topographic lowlands.

Flow lines, paths followed by particles of water as they move through the aquifer in the direction of decreasing head, were drawn at right angles to the piezometric surface contours to define the area of diversion. As measured from figure 85, the area of diversion is about 58 square miles.

The piezometric surface map of the Silurian dolomite aquifer was compared with water-level data for the period prior to development, and water-level changes were computed. The greatest declines in the piezometric surface occurred in the immediate vicinity of Libertyville and averaged about 85 feet.

Data on water levels in shallow sand and gravel aquifers given in table 24 indicate that the piezometric surface of the shallow sand and gravel aquifers more closely resembles

Table 24. Water-Level Data for Wells in Shallow Sand and Gravel Aquifers in Libertyville Area

Well number	Owner	Depth (ft)	Depth to water (ft below measuring point)	Land surface elev. (ft above MSL)	Water level elev. (ft above MSL)	Date measured
LKE—						
3N10E-						
4.7h	J. L. Smith	24	14.7	875	860	7/6/62
3N11E-						
4.1h	Daughters of Charity	50	16.1	677	661	8/16/62
5.8h	Diamond Lake Cmty.	32	11.70	722	710	4/30/62
5.6g	W. Martin	55	15.09	722	707	7/26/62
15.2c	Do-Mor Day Camp	55	9	650	641	6/11/62
15.4c	Dove	38	10.85	650	639	3/29/62
4N9E-						
2.1d	L. H. Wood	—	18.5	785	765	6/25/62
3.3e	J. McNally	30	5	790	785	6/25/62
10.3g	E. Kulin	60	15.4	780	765	6/26/62
12.2b1	W. Roney	60	21.2	805	784	7/17/62
15.2e	Fisher	60	8.8	772	763	8/15/62
33.7c	M. Snider	26	6	741	735	7/3/62
4N10E-						
30.4e	Ascension Cmty.	15	10.0	778	768	7/12/62
4N11E-						
12.5h	—	40	22.24	710	688	4/19/62
17.8e	C. Simmonds	23	4.30	740	736	4/16/62
28.3g1	Florsheim Estate	55	28.55	708	679	4/30/62
4N12E-						
18.5a	W. R. Winters	60	33.4	688	655	8/10/62
5N10E-						
32.8c1	H. Vanderspool	—	75.6	812	736	6/21/62
5N11E-						
34.4a1	E. S. Richardson	27	21.20	713	692	6/18/62
MCH—						
4N9E-						
8.6f	W. Krepel	40	27.5	765	725	6/29/62
17.3f	J. J. Morinich	65	10.9	755	744	8/15/62
18.1g	Holiday Hills Inc.	—	21.4	755	734	6/29/62

the topography than does the piezometric surface of the Silurian dolomite aquifer. The data also indicate that the piezometric surface of shallow sand and gravel aquifers is at most places at a higher elevation than the piezometric surface of the Silurian dolomite aquifer.

Coefficient of Transmissibility of Silurian Dolomite Aquifer at Libertyville

The coefficients of transmissibility determined from well-production data pertain to parts of the Silurian dolomite aquifer in the immediate vicinity of production wells and may not be representative of the regional coefficient of transmissibility of the Silurian dolomite aquifer. Flow-net analysis of the piezometric surface was made to determine the average coefficient of transmissibility of the part of the aquifer in the deep cone of depression at Libertyville. The area enclosed by the contour line having an elevation of 620 feet near Libertyville was selected for analysis (see figure 85).

From Darcy's equation

$$T = Q/IL \qquad (14)$$

where:

T = coefficient of transmissibility, in gpd/ft
Q = discharge, in gpd
I = hydraulic gradient, in ft/mi
L = width of flow cross section, in mi

The quantity of water, Q, moving across the 620-foot contour line is equal to the total pumpage (1.25 mgd) from the Silurian dolomite aquifer in the Libertyville area minus the water taken from storage and derived from vertical leakage within the area enclosed by the 620-foot contour line. The amount of water taken from storage is very small; however, the amount of vertical leakage into the cone of depression was estimated to be about 40,000 gpd, on the basis of water-level data and the average recharge rate for the Libertyville area. Thus, Q is about 1.21 mgd. The hydraulic gradient, I, and the length of flow cross section, L, at the 620-foot contour line were scaled from figure 85. Computations made using the data mentioned above and equation 14 indicate that the average coefficient of transmissibility of the part of the Silurian dolomite aquifer within the Libertyville cone of depression is 9500 gpd/ft. This value compares favorably with the average coefficient of transmissibility computed from specific-capacity data.

Recharge to Aquifers

Recharge to aquifers in the Libertyville area occurs locally as vertical leakage of water through clayey deposits, and has precipitation as its source. Vertical movement is possible because of the large differentials in head between the water table in the shallow sand and gravel aquifers and the piezometric surface of the Silurian dolomite aquifer. The rate of recharge to the Silurian dolomite aquifer was estimated using the piezometric surface map and past records of pumpage and water levels.

The area of diversion of production wells in the Libertyville area was delineated with the piezometric surface map in figure 85. The water levels in the dolomite aquifer and overlying sand and gravel aquifers vary greatly from place to place and from time to time, mostly because of the shifting of pumpage from well to well and variations in total well field pumpage. At no location, however, is there any apparent continuous decline that cannot be explained by pumpage increases. Within a relatively short time after each increase in pumpage, recharge from vertical leakage through the glacial drift increased in proportion to pumpage as vertical hydraulic gradients became greater and the area of diversion expanded. Therefore, recharge to the Silurian dolomite aquifer and deeply buried sand and gravel aquifers within the area of diversion is equal to the total pumpage from these aquifers, or about 3 mgd in 1962.

The quotient of the quantity of leakage (recharge) and the area of diversion is the rate of recharge. The area of

diversion is about 58 square miles; therefore, the recharge rate to the Silurian dolomite aquifer was about 52,000 gpd/sq mi in 1962.

Darcy's equation indicates that the recharge rate varies with the vertical head loss ($\triangle h$) associated with leakage of water through the confining bed. The average vertical head loss in 1962 was computed to be about 40 feet by comparing the piezometric surface map for the Silurian dolomite aquifer with water-level data for wells in shallow sand and gravel deposits (source bed for the Silurian dolomite aquifer). The average recharge rate taking into account head loss is about 1300 gpd/sq mi/ft. Data were not sufficient to evaluate the recharge rate for shallow sand and gravel aquifers.

Vertical Permeability of Confining Bed

Based on Darcy's equation, the vertical permeability of the confining bed between the shallow sand and gravel aquifers and the Silurian dolomite and deeply buried sand and gravel aquifers may be computed by multiplying the recharge rate per unit area per foot of head loss ($Q_c/\triangle h A_c$) by the saturated thickness of the confining bed. Based on available well logs, the average saturated thickness of the glacial drift confining bed within the area of diversion is about 200 feet. It is possible that shaly beds in the upper part of the Silurian dolomite aquifer may also retard vertical movement of water towards permeable zones within the dolomite aquifer. A coefficient of vertical permeability of 0.009 gpd/sq ft was computed by substituting appropriate data in Darcy's equation. The coefficient of vertical permeability based on the piezometric surface map applies to the entire thickness of the confining bed between the shallow sand and gravel aquifers and the Silurian dolomite and deeply-buried sand and gravel aquifers.

Practical Sustained Yield of Existing Well Fields at Libertyville and Mundelein

Silurian Dolomite Aquifer

Because the Silurian dolomite aquifer is thick, deeply buried, and on a regional basis has moderate permeabilities and great areal extent, cones of depression of production wells can extend for considerable distances and available water resources can be developed with a reasonably small number of wells and well fields. There are large areas outside present areas of diversion that are not influenced by present pumpage, and water levels in dolomite wells are not at critical stages, suggesting that the practical sustained yield of the existing well fields is much greater than present withdrawals.

Areas influenced by pumping include sites where the Silurian dolomite aquifer yields very little water to individual wells. In addition, the piezometric surface map is regular in appearance and could be favorably compared to piezometric surface maps for uniform sand and gravel or sandstone aquifers. These facts indicate that the inconsistency of the Silurian dolomite aquifer has little effect on the regional response of the aquifer to pumping and should not seriously deter the full development of available ground-water resources.

In 1962 large parts of the Libertyville area were influenced by pumping from the Silurian dolomite aquifer. Many pumping centers are so closely spaced that individual cones of depression overlap and there is competition between pumping centers. Interference between pumping centers affects values of discharge and drawdown in individual wells. This situation is particularly apparent in the vicinity of Libertyville and Mundelein.

The pumping levels in dolomite wells in the area of diversion are well above the top of the Silurian dolomite aquifer, and there is available drawdown to support future pumpage increases. When nonpumping levels recede to stages below the top of the Silurian dolomite aquifer the yields of production wells will decrease and become critical for two reasons: 1) the aquifer will be partially dewatered, thus decreasing the coefficient of transmissibility; and 2) based on a recent study by Zeizel et al. (1962), well loss in dolomite wells increases at an accelerating rate when nonpumping levels recede to stages below the top of the Silurian dolomite aquifer. Therefore, the practical sustained yield of existing well fields is limited by available drawdown to the top of the Silurian dolomite aquifer.

Drawdowns available for future increases in pumpage were estimated for Libertyville and Mundelein from the piezometric surface and the bedrock topography maps. It was assumed that critical water levels will result if nonpumping levels are below the top of the Silurian dolomite aquifer. The amounts of water, in addition to withdrawals in 1962, that can be withdrawn from the Libertyville and Mundelein pumping centers without creating critical water-level conditions were estimated as the products of available drawdown and the average yield of the dolomite aquifer given in figure 73. Estimated additional withdrawals were added to pumping rates in 1962 to obtain the practical sustained yield of existing well fields.

Computations indicate that the practical sustained yield of the dolomite wells at Mundelein is about 1.3 mgd, or about 1.1 mgd more than the average annual rate of pumpage from wells in 1962. The practical sustained yield of existing dolomite wells at Libertyville is about 2.0 mgd, or about 1.0 mgd more than the average annual rate of pumpage from wells in 1962.

In order to increase the amount of recharge to the Silurian dolomite aquifer from the 1962 rate to the practical sustained yield, the product $\triangle h A_c$ must increase in direct proportion to the increase in pumpage. Thus, full development of the practical sustained yield will be accompanied by increases in the area of diversion and water-level lowering.

Glacial Drift Aquifer

Mundelein has three wells penetrating sand and gravel aquifers. The majority of ground-water withdrawals in 1962 at Mundelein were from these wells. Geologic and hydrologic data are not available to predict with a high degree of accuracy the practical sustained yield of these wells. However, based on pumping-level data in table 22 the practical sustained yield of these wells has already been exceeded. Pumping levels have been be: 's of screens. Exposing screens to air often accelerates   .: ate of clogging of screen openings and is undesirab.e. Thus, present pumping rates are excessive in these wells, and the practical sustained yield is slightly less than the average annual rate of withdrawal in 1962. A reasonable estimate of the practical sustained yield of the three sand and gravel wells based upon 1962 water-level data and allowable drawdown to the tops of screens is 0.75 mgd.

CHICAGO HEIGHTS AREA

Water for municipal use in Chicago Heights and Park Forest is obtained locally from wells in a shallow dolomite aquifer. Since 1900 the average daily withdrawal from the two municipal water supplies steadily increased from 700,000 gallons to 7.84 million gallons in 1962. Continual increases in pumpage caused water levels to decline about 90 feet at Chicago Heights and about 30 feet at Park Forest. Water levels in dolomite wells are not yet at critical stages at Chicago Heights or Park Forest; however, water levels in dolomite wells in the immediate vicinity of Chicago Heights were below the top of the dolomite in 1962. Available data indicate that the dolomite aquifer is capable of yielding more water than is being withdrawn at present.

Geography and Climate

Chicago Heights is located in southeastern Cook County about 27 miles south of the Chicago loop. Detailed study was confined to a square area, hereafter referred to as "the Chicago Heights area," of about 150 square miles. The area is located in southern Cook County and eastern Will County, as shown in figure 86, and is between 87° 31' and 87° 45' west longitude and between 41° 20' and 41° 35' north latitude.

Chicago Heights lies near the center of the area. Other cities and villages within the area are: East Chicago Heights, Crete, Flossmoor, Homewood, Steger, South Chicago Heights, Park Forest, Matteson, Richton Park, Olympia Fields, Sauk Village, Glenwood, Thornton, and Monee. State highway 1 and U.S. 30 and 54 pass through the area as do the Illinois Central, the New York Central, and the Chicago and Eastern Illinois railroads.

The Chicago Heights area lies in the Central Lowland Physiographic Province. The land surface is characterized by relatively flat terrain; extensive surface and subsurface drainage is necessary for development. The average land surface elevation declines from about 750 feet in the southern part of the Chicago Heights area to about 630 feet in the northeastern part.

Drainage is largely northeastward to tributaries of the Little Calumet River flowing in a course about 8 miles north of Chicago Heights. Butterfield Creek and a part of Thorn Creek drain the western portion of the area: Deer Creek and North Creek drain most of the eastern part of the area.
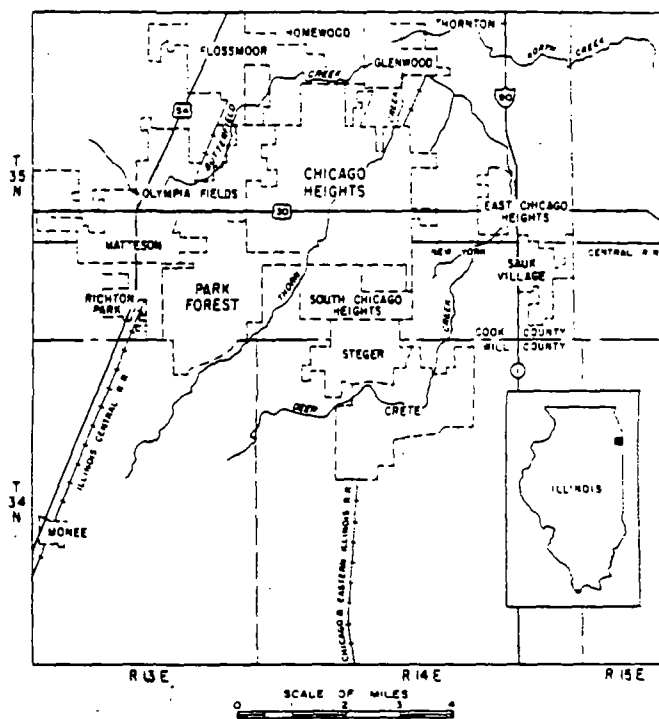
Figure 86. Location of Chicago Heights area

Graphs of annual and mean monthly precipitation given in figures 87 and 88 were compiled from precipitation data collected by the Corps of Engineers at Brandon Road Dam near Joliet, about 22 miles west of Chicago Heights. According to these records the mean annual precipitation is 33.65 inches. On the average, the months of greatest precipitation are May, June, and September, each having more than 3.5 inches; January, February, and December are the months of least precipitation, each having less than 2 inches.

The Chicago Heights area experienced a severe drought beginning in 1912. For the period 1912 through 1926, cumulative deficiency of precipitation at Chicago Heights was about 56 inches. Recharge from precipitation was much

57

Public Groundwater Supplies

in Lake County

by DOROTHY M. WOLLER and JAMES P. GIBB

Figure 2. Location of public groundwater supply systems in Lake County

Groundwater Development for Public Use

Groundwater is used as a source of public water supply at 23 municipalities, 32 subdivisions, 2 state parks, and 1 treatment plant in Lake County. The locations of these supplies are shown in figure 2.

Sand and gravel deposits in the unconsolidated materials above bedrock are tapped by 24 public water systems in Lake County as a source of all or part of their water supply. There are presently 52 production and standby wells, ranging in depth from 35 to 292 ft, tapping only the sand and gravel

deposits. Their reported yields range from 10 to 1500 gpm depending primarily upon the type of well and the permeability, thickness, and areal extent of the sand and gravel unit tapped by each well. Production from these wells for 1972 through 1975 was estimated to be about 3,330,000 gpd.

The analyses of water from these wells show that the iron content ranges from 0.0 to 2.5 mg/l and the hardness from 118 to 580 mg/l. The sulfate content of water from

4

of McMillen Drive, approximately 350 ft N and 1500 ft W of the SE corner of Section 8, T46N, R10E. The land surface elevation at the well is approximately 770 ft.

A drillers log of Well No. 4 follows:

Strata	Thickness (ft)	Depth (ft)
Fill	3	3
Soft sandy yellow clay	5	8
Sand and gravel and boulders	16	24
Soft sticky gray clay, some thin sand streaks	56	80
Fine to coarse sand and gray clay	5	85
Fine gray sand	5	90
Blue clay	4	94
Very fine gray sand	8	102
Soft gray clay	3	105
Medium fine to coarse sand, some gravel and boulders 115 to 121 ft very coarse	16	121
Medium fine to coarse sand, gravel and boulders, not as much coarse stuff, also not as tight	4	125
Very coarse sand and gravel, some fine showing at 129 ft	4	129
Very fine gray sand	12	141

A 34-in. diameter hole was drilled to a depth of 15 ft, reduced to 30 in. between 15 and 26 ft, and finished 28 in. in diameter from 26 to 141 ft. The well is cased with 12-in. welded steel pipe from 2 ft above land surface to a depth of 109 ft followed by 20 ft of 12-in. No. 5 (0.105 in.) Layne stainless steel shutter screen. The annulus between the bore hole and casing-screen assembly is filled with cement grout from 0 to 40 ft, with pea gravel from 40 to 86 ft, and with Muscatine No. 3 gravel from 86 to 141 ft.

A production test using one observation well was conducted by the driller on June 22, 1965. After 8 hr of pumping at a rate of 632 gpm, the drawdown was 12 ft from a non-pumping water level of 32 ft below land surface.

On March 1, 1967, the well reportedly produced 800 gpm with a drawdown of 15 ft from a nonpumping water level of 34 ft.

In December 1970, the well reportedly produced 825 gpm

with a drawdown of 14 ft from a nonpumping level of 33 ft.

The pumping equipment presently installed is a Jacuzzi oil-lubricated turbine pump (Model No. 10 6T-490) set at 75 ft, rated at 775 gpm at about 175 ft and powered by a 60-hp General Electric motor (Model No. 5K6257XHIA, Serial No. KAJ1006465).

The following mineral analysis made by the Illinois Environmental Protection Agency (Lab. No. B34770) is for a water sample from the well collected March 1, 1976, after 2 hr of pumping at 750 gpm.

WELL NO. 4, LABORATORY NO. B34770

		mg/l	me/l			mg/l	me/l
Iron	Fe	0.7		Silica	SiO$_2$	21	
Manganese	Mn	0.00		Fluoride	F	0.7	0.04
Ammonium	NH$_4$	1.2	0.07	Boron	B	0.4	
Sodium	Na	36	1.57	Nitrate	NO$_3$	0	0.00
Potassium	K	1.5	0.04	Chloride	Cl	4.5	0.13
Calcium	Ca	43	2.15	Sulfate	SO$_4$	43	0.89
Magnesium	Mg	29	2.39	Alkalinity (as CaCO$_3$)		256	5.12
Arsenic	As	0.00					
Barium	Ba	0.1		Hardness (as CaCO$_3$)		226	4.52
Copper	Cu	0.01					
Cadmium	Cd	0.00		Total dissolved			
Chromium	Cr	0.00		minerals		329	
Lead	Pb	0.00					
Mercury	Hg	0.0000		pH (as rec'd)	8.4		
Nickel	Ni	0.0		Radioactivity			
Selenium	Se	0.00		Alpha pc/l	1.2		
Silver	Ag	0.00		±deviation	1.2		
Cyanide	CN	0.00		Beta pc/l	1.7		
Zinc	Zn	0.0		±deviation	1.2		

A 5-in. diameter test hole was constructed in July 1975 to a depth of 228 ft by the J. P. Miller Artesian Well Co., Brookfield. The test hole was located approximately 1320 ft S and 1250 ft W of the NE corner of Section 17, T46N, R10E. Upon completion, the nonpumping water level was reported to be 43 ft below land surface.

ARDEN SHORES ESTATES SUBDIVISION

Arden Shores Estates Subdivision (est. 67), located approximately 1 mile northwest of Lake Bluff, installed a public water supply in 1953. The water system is owned and operated by the Arden Shores Civic Improvement Association. One well is in use. In 1972 there were 19 services, none metered; the estimated average daily pumpage was 5000 gpd. The water is not treated. The natural fluoride concentration in the water is adequate to satisfy state requirements.

WELL NO. 1, finished in Silurian dolomite, was completed in June 1954 to a depth of 283 ft by Paul Neely, Batavia. The well is located on an unoccupied lot at 237 Bay Shore Drive, approximately 1000 ft S and 2300 ft W of the NE corner of Section 17, T44N, R12E. The land surface eleva-

tion at the well is approximately 693 ft.

The well is cased with 6-in. pipe from 1 ft above the floor of a 6-ft deep pit to an unknown depth.

In 1957, the well reportedly produced 30 gpm with a drawdown of 90 ft from a nonpumping water level of 33 ft.

The pumping equipment presently installed is a Sta-Rite submersible pump (Model No. 20P4F2) set at 189 ft, rated at 20 gpm at about 250 ft TDH, and powered by a 1 1/2-hp 3450 rpm electric motor.

The following mineral analysis made by the Illinois Environmental Protection Agency (Lab. No. B37397) is for a water sample from the well collected March 22, 1976.

		mg/l	me/l			mg/l	me/l
Iron	Fe	0.1		Silica	SiO$_2$	13	
Manganese	Mn	0.02		Fluoride	F	0.8	0.04
Ammonium	NH$_4$	0.28	0.02	Boron	B	0.7	
Sodium	Na	75	3.26	Nitrate	NO$_3$	0.5	0.01
Potassium	K	1.1	0.03	Chloride	Cl	7.5	0.21
Calcium	Ca	28	1.40	Sulfate	SO$_4$	190	3.95
Magnesium	Mg	19	1.56	Alkalinity (as CaCO$_3$)		112	2.24
Arsenic	As	0.00					
Barium	Ba	0.1		Hardness (as CaCO$_3$)		148	2.96
Copper	Cu	0.01					
Cadmium	Cd	0.00		Total dissolved			
Chromium	Cr	0.00		minerals		382	
Lead	Pb	0.00					
Mercury	Hg	0.0000		pH (as rec'd)		7.9	
Nickel	Ni	0.0		Radioactivity			
Selenium	Se	0.0		Alpha pc/l		1.2	
Silver	Ag	0.00		±deviation		1.2	
Cyanide	CN	0.00		Beta pc/l		1.7	
Zinc	Zn	0.0		±deviation		1.4	

BARRINGTON

The village of Barrington (7701) installed a public water supply in 1898. This village also extends into Cook County and two of the wells are located there. Four wells are in use. This supply is cross connected with the Jewel Companies, Inc., and the Quaker Oats Co. In 1950 there were 1321 services; the average and maximum daily pumpages were 500,000 and 850,000 gpd, respectively. In 1973 there were 3110 services, all metered; the average and maximum daily pumpages were 1,170,000 and 2,170,000 gpd, respectively. The water is aerated, chlorinated, and fluoridated.

WELL NO. 1, finished in Silurian dolomite, was completed in 1898 to a depth of 305 ft. The well is located in the rear of the village hall near Hough and Station Sts., approximately 425 ft S and 1200 ft E of the NW corner of Section 1, T42N, R9E, Cook County. The land surface elevation at the well is approximately 830 ft.

A drillers log of Well No. 1 follows:

Strata	Thickness (ft)	Depth (ft)
Drift	200	200
Lime rock	105	305

A 12-in. diameter hole was drilled to a depth of 200 ft and finished 10 in. in diameter from 200 to 305 ft. The well is cased with 12-in. ID pipe from 0.5 ft above the pumphouse floor to a depth of 200 ft.

On November 9, 1922, after a 12-hr idle period, the well reportedly produced 270 gpm for 9 hr with a drawdown of 5 ft from a nonpumping water level of 56 ft below the pumphouse floor (3.5 ft above top of casing).

In 1923, after pumping at a rate of 400 gpm, the drawdown was 16 ft from a nonpumping water level of 60 ft.

On June 21, 1928, after pumping at a rate of 380 gpm, the drawdown was 16 ft from a nonpumping water level of 61 ft.

On November 7, 1933, the well reportedly produced 350 gpm for 4.5 hr with a drawdown of 3.65 ft from a nonpumping water level of 60.77 ft below the pumphouse floor.

On July 20, 1943, the nonpumping water level was reported to be 66.1 ft below the pump base when Well No. 2 was pumping.

In 1962, the nonpumping water level was reported to be 90 ft.

In 1973, after the Henry Boysen Co., Libertyville, installed new pump bowls, the well reportedly produced 800 gpm with a drawdown of 7 ft from a nonpumping water level of 108 ft.

The pumping equipment presently installed is a 2-stage Layne & Bowler turbine pump (Serial No. 24887) set at 130 ft, rated at 850 gpm, and powered by a 50-hp U.S. electric motor (Serial No. 917501).

The following mineral analysis made by the Illinois Environmental Protection Agency (Lab. No. C007451) is for a water sample from the well collected April 22, 1974, after 30 min of pumping at 850 gpm.

		mg/l	me/l			mg/l	me/l
Iron	Fe	0.4		Silica	SiO$_2$	27.0	
Manganese	Mn	0.00		Fluoride	F	0.6	0.03
Ammonium	NH$_4$	0.84	0.05	Boron	B	0.2	
Sodium	Na	18	0.78	Nitrate	NO$_3$	0.2	0.00
Potassium	K	1.6	0.04	Chloride	Cl	2	0.06
Calcium	Ca	55	2.74	Sulfate	SO$_4$	71	1.48
Magnesium	Mg	41	3.37	Alkalinity (as CaCO$_3$)		286	5.72
Arsenic	As	0.00					
Barium	Ba	0.0		Hardness (as CaCO$_3$)		306	6.12
Copper	Cu	0.00					
Cadmium	Cd	0.00		Total dissolved			
Chromium	Cr	0.00		minerals		402	
Lead	Pb	0.00					
Mercury	Hg	0.0000		pH (as rec'd)		8.1	
Nickel	Ni	0.0		Radioactivity			
Selenium	Se	0.00		Alpha pc/l		0.7	
Silver	Ag	0.00		±deviation		1.2	
Cyanide	CN	0.00		Beta pc/l		2.1	
Zinc	Zn	0.00		±deviation		1.8	

Countryside Estates Subdivision (est. 274), located 2 miles north of Gurnee, installed a public water supply in 1957. The water system is owned and operated by the Charmar Water Co. of Utilities, Inc. Two wells are in use. In 1961 there were 50 services, all metered. In 1974 there were 76 services, all metered; the average and maximum daily pumpages were 11,900 and 18,000 gpd, respectively. The water from Well No. 2 is chlorinated. The natural fluoride concentration in the water is adequate to satisfy state requirements.

WELL NO. 1, finished in sand and gravel, was completed in March 1956 to a depth of 208 ft by the Hoover Water Well Service, Zion. The well is located between Delany Road and Shirley Drive, approximately 665 ft N and 280 ft W of the SE corner of Section 2, T45N, R11E. The land surface elevation at the well is approximately 680 ft.

A correlated drillers log of Well No. 1 furnished by the State Geological Survey follows:

Strata	Thickness (ft)	Depth (ft)
PLEISTOCENE SERIES		
Yellow clay	10	10
Soft blue clay	75	85
Blue hardpan	55	140
Blue clay	61	201
Mixed sand and gravel fine to coarse	3	204
No record	4	208

The following mineral analysis made by the Illinois Environmental Protection Agency (Lab. No. 02432) is for a water sample from the well collected October 25, 1971, after 30 min of pumping.

WELL NO. 1, LABORATORY NO. 02432

		mg/l	me/l			mg/l	me/l
Iron	Fe	0.0		Silica	SiO₂	11	
Manganese	Mn	0.0		Fluoride	F	0.94	0.05
Ammonium	NH₄	0.26	0.01	Boron	B	0.3	
Sodium	Na	100	4.35	Nitrate	NO₃	0.88	0.01
Potassium	K	0.8	0.02	Chloride	Cl	9.5	0.27
Calcium	Ca	24	1.20	Sulfate	SO₄	225	4.68
Magnesium	Mg	15	1.23	Alkalinity (as CaCO₃)		88	1.76
				Hardness (as CaCO₃)		120	
Barium	Ba	0.0		Total dissolved			
Copper	Cu	0.0		minerals		432	
Cadmium	Cd	0.00					
Chromium	Cr	0.0		pH (as rec'd)		8.2	
Lead	Pb	0.00		Radioactivity			
Mercury	Hg	<0.0005		Alpha pc/l		0	
Nickel	Ni	0.05		±deviation		0	
Silver	Ag	0.0		Beta pc/l		1	
Zinc	Zn	0.0		±deviation		0	

A 5-in. diameter hole was drilled to a depth of 208 ft. The well is cased with 5-in. galvanized pipe from 1.7 ft above the concrete floor of a 4-ft deep pit to a depth of 204 ft and equipped with 4.7 ft (4 ft exposed) of 4.8-in.

No. 30 slot Johnson silicon brass screen. The top of the well casing is equipped with a pitless adapter.

Upon completion, the well reportedly produced 20 gpm with a drawdown of 21 ft from a nonpumping water level of 42 ft below the top of the casing.

The pumping equipment presently installed consists of a 1 1/2-hp U.S. electric motor, a Red Jacket submersible pump set at 157 ft, rated at 25 gpm at about 70 ft head, and has 157 ft of 1.2-in. column pipe.

WELL NO. 2, finished in Silurian dolomite, was completed in April 1957 to a depth of 285 ft by L. J. Watson, Harvey. The well is located 300 ft north of Well No. 1, approximately 965 ft N and 260 ft W of the SE corner of Section 2, T45N, R11E. The land surface elevation at the well is approximately 675 ft.

A drillers log of Well No. 2 follows:

Strata	Thickness (ft)	Depth (ft)
Shale, sand and gravel	210	210
Lime	75	285

A 6-in. diameter hole was drilled to a depth of 285 ft. The well is cased with 6-in. pipe from above the roof of a 4-ft deep concrete pit to a depth of 210 ft. The top of the well casing is equipped with a pitless adapter.

Upon completion, the well reportedly produced 22 gpm with a drawdown of 175 ft from a nonpumping water level of 25 ft below the top of the casing.

The pumping equipment presently installed is a Red Jacket submersible pump set at 232 ft, and powered by a 1 1/2-hp U.S. electric motor.

The following mineral analysis made by the Illinois Environmental Protection Agency (Lab. No. 02431) is for a water sample from the well collected October 25, 1971, after 30 min of pumping.

WELL NO. 2, LABORATORY NO. 02431

		mg/l	me/l			mg/l	me/l
Iron	Fe	0.0		Silica	SiO₂	11	
Manganese	Mn	0.0		Fluoride	F	0.9	0.05
Ammonium	NH₄	0.3	0.01	Boron	B	0.9	
Sodium	Na	110	4.78	Nitrate	NO₃	0.9	0.01
Potassium	K	0.8	0.02	Chloride	Cl	9.8	0.28
Calcium	Ca	24	1.20	Sulfate	SO₄	225	4.68
Magnesium	Mg	15	1.23	Alkalinity (as CaCO₃)		88	1.76
				Hardness (as CaCO₃)		120	
Barium	Ba	0.0		Total dissolved			
Copper	Cu	0.0		minerals		433	
Cadmium	Cd	0.00					
Chromium	Cr	0.0		pH (as rec'd)		8.2	
Lead	Pb	0.00		Radioactivity			
Mercury	Hg	<0.0005		Alpha pc/l		0	
Nickel	Ni	0.0		±deviation		1	
Silver	Ag	0.0		Beta pc/l		2	
Zinc	Zn	0.0		±deviation		2	

with 16.8 ft of 8-in. Cook screen. The screened section consists of 6 ft of No. 10 slot followed by 10.8 ft of No. 14 slot. The upper 6 ft of the well casing is cemented in a 10-in. pipe for protection against surface pollution.

Upon completion, the well reportedly produced 281 gpm with a drawdown of 17 ft from a nonpumping water level of 72 ft below land surface.

The pumping equipment presently installed is a Byron Jackson oil-lubricated turbine pump (Model No. OKHC-9STG) set at 95 ft, rated at 200 gpm against 180 ft head, and powered by a 15-hp 1800 rpm U.S. electric motor (Serial No. 2362563). The well is equipped with 95 ft of airline.

The following mineral analysis made by the Illinois Environmental Protection Agency (Lab. No. 04508) is for a water sample from the well collected March 15, 1972, after 10 min of pumping at 200 gpm.

WELL NO. 2, LABORATORY NO. 04508

		mg/l	me/l			mg/l	me/l
Iron	Fe	0.3	0.01	Silica	SiO$_2$	22	
Manganese	Mn	0.0		Fluoride	F	0.4	0.02
Ammonium	NH$_4$	0.1	0.01	Boron	B	0.1	
Sodium	Na	2.5	0.11	Nitrate	NO$_3$	0.0	
Potassium	K	0.9	0.02	Chloride	Cl	3.6	0.10
Calcium	Ca	88	4.39	Sulfate	SO$_4$	40	0.83
Magnesium	Mg	44	3.62	Alkalinity (as CaCO$_3$)		344	6.88
Barium	Ba	0.0		Hardness (as CaCO$_3$)		392	
Copper	Cu	0.1		Total dissolved			
Cadmium	Cd	0.00		minerals		485	
Chromium	Cr	0.0		pH (as rec'd)	7.3		
Lead	Pb	0.00		Radioactivity			
Mercury	Hg	<0.0005		Alpha pc/l	0		
Nickel	Ni	0.0		±deviation	0		
Silver	Ag	0.0		Beta pc/l	0		
Zinc	Zn	0.15		±deviation	1		

ILLINOIS BEACH STATE PARK

Illinois Beach State Park, located along Lake Michigan in the northern part of Lake County, installed a public water supply in 1947. Water was obtained from a deep well until 1965 when the park area water system was connected to the Zion–Benton Treatment Plant. The well is disconnected from the water system but is available for use at a beach side fish hatchery. In 1974 the average and maximum daily consumption rates for the park were 20,386 and 30,000 gpd, respectively.

WELL NO. 1, open to the Silurian dolomite, Galena-Platteville Dolomite, and the Glenwood–St. Peter Sandstone, was constructed in April 1947 to a depth of 160 ft and deepened in August 1947 to 1002 ft (measured in 1972 at 964 ft deep) by the S. B. Geiger & Co., Chicago. The well is located just south of Zion, about 300 ft from Lake Michigan in line with Beach Road extended, approximately 700 ft N and 500 ft E of the SW corner of Section 26, T46N, R12E. The land surface elevation at the well is approximately 585 ft.

A sample study summary log of Well No. 1 furnished by the State Geological Survey follows:

Strata	Thickness (ft)	Depth (ft)
PLEISTOCENE SERIES		
Sand, yellowish brown	25	25
Till, sandy, gravelly, dark yellowish brown	25	50
Till, pinkish brown	40	90
Gravel, light gray	5	95
Till, calcareous, pinkish brown	20	115
No sample	5	120
SILURIAN SYSTEM		
Niagaran Series		
Dolomite, white to yellowish gray; cherty in lower portion	150	270
Alexandrian Series		
Dolomite, white to yellowish gray, cherty at top	25	295

Strata (continued)	Thickness (ft)	Depth (ft)
ORDOVICIAN SYSTEM		
Maquoketa Group		
Shale, dolomitic, green; some dolomite streak at top	200	495
Galena Group		
Dolomite, sandy, pale brown to buff, some yellowish gray at top	155	650
Dolomite, brown to gray	38	688
Platteville Group		
Dolomite, brownish to gray	142	830
Ancell Group		
Glenwood Formation		
Dolomite buff to brown	45	875
Sandstone, white dolomitic, fine to coarse	25	900
St. Peter Sandstone		
Sandstone, yellowish white, fine to coarse, incoherent	102	1002

A 12-in. diameter hole was drilled to a depth of 124 ft, reduced to 8 in. between 124 and 440 ft, and finished 6 in. in diameter from 440 to 1002 ft. The well is cased with 8-in. pipe from land surface to a depth of 124 ft and a 6-in. ID liner from 290 to 440 ft.

A production test was conducted on April 30, 1947, by representatives of the driller, the State Water Survey, and the Illinois State Division of Architecture and Engineering, when the well was 160 ft deep and cased with 8-in. pipe to 124 ft. After 5 hr of pumping at rates of 9.2 to 19.2 gpm, the final drawdown was 128 ft from a nonpumping water level of 17 ft below land surface. Fifteen min after pumping was stopped, the water level had recovered to 77 ft.

A second production test was conducted after deepening on August 18, 1947, by representatives of the driller, the State Water Survey, and the Illinois State Division of Architecture and Engineering. After 2.7 hr of pumping at a rate of 33 gpm, the drawdown was 96.5 ft from a nonpumping

water level of 12.0 ft below the top of the casing. Pumping continued intermittently for 3.7 additional hr at a rate of about 40 gpm. After the pumping was stopped, the water level recovered to 15.0 ft in 1.5 hr.

In 1972, the well reportedly produced 35 gpm continuously for 2 months with a drawdown of 478 ft from a nonpumping water level of 22 ft.

The pumping equipment presently installed is a Sta-Rite submersible pump (Model No. 90P6M3-6) set at 510 ft, rated at 54 gpm at about 450 ft TDH, and powered by a 15-hp Sta-Rite electric motor.

The following mineral analysis (Lab. No. 144672) is for a water sample from the well collected October 3, 1957.

WELL NO. 1, LABORATORY NO. 144672

		mg/l	me/l			mg/l	me/l
Iron (total)	Fe	0.6		Silica	SiO_2	47.5	
Manganese	Mn	0.0		Fluoride	F	0.8	
Calcium	Ca	28.8	1.44	Boron	B	1.0	
Magnesium	Mg	12.4	1.01	Chloride	Cl	27	0.76
Ammonium	NH_4	Tr	Tr	Nitrate	NO_3	0.4	0.01
Sodium	Na	123	5.35	Sulfate	SO_4	222.5	4.63
Turbidity		Tr		Alkalinity (as $CaCO_3$)		120	2.40
Color		0		Hardness (as $CaCO_3$)		123	2.45
Odor		0		Total dissolved		-	
Temp.		52.7 F (reported)		minerals		525	

ISLAND LAKE

The village of Island Lake (1973) installed a public water supply in 1940. This village also extends into McHenry County and two of the wells are located there. The water system is owned and operated by the Island Lake Water Co. Three wells (Nos. 1, 2, and 3) are in use. In 1952 there were 450 services, 350 were metered. In 1973 there were 580 services, all metered; the average and maximum daily pumpages were 47,022 and 70,000 gpd, respectively. The water is chlorinated and fluoridated.

WELL NO. 1 (Well 19-U), finished in sand and gravel, was completed in July 1940 to a depth of 116 ft (effective depth 115 ft) by Henry Boysen, Jr., Libertyville. The well is located at the corner of Midway and Fairfield Drives, approximately 1130 ft N and 190 ft E of the SW corner of Section 21, T44N, R9E, Lake County. The land surface elevation at the well is approximately 770 ft.

A drillers log of Well No. 1 follows:

Strata	Thickness (ft)	Depth (ft)
Yellow stoney gravel	40	40
Dirty gravel and sand	51	91
Gravel and sand	25	116

A 10-in. diameter hole was drilled to a depth of 116 ft. The well is cased with 10-in wrought iron pipe from 1.2 ft above the floor of a 12-ft deep pit to a depth of 92 ft followed by 24 ft of 9.6-in. Cook screen. The screened section from top to bottom consists of 5 ft of No. 60 slot, 10 ft of No. 14 slot, and 8 ft of No. 40 slot with 1 ft of blank section at the bottom.

Upon completion, the well reportedly produced 503 gpm for 8 hr with a drawdown of 11 ft from a nonpumping water level of 29 ft below land surface.

On October 27, 1959, the nonpumping water level was reported to be 26 ft below land surface.

The well was acidized in 1960 by the Dow Chemical Co. and the yield was reportedly improved from 115 to 435 gpm.

On May 20, 1963, the nonpumping water level was reported to be 30 ft.

The pumping equipment presently installed consists of a 20-hp 1800 rpm U.S. electric motor, an 8-in., 11-stage Aurora turbine pump (No. 69213) set at 90 ft, rated at 200 gpm at about 250 ft TDH, and has 90 ft of 5-in. column pipe. The well is equipped with 90 ft of airline.

A mineral analysis made by the Illinois Environmental Protection Agency (Lab. No. C004680) of a sample collected December 18, 1973, after pumping for 30 min at 300 gpm, showed the water to have a hardness of 397 mg/l, total dissolved minerals of 466 mg/l, and an iron content of 1.4 mg/l.

WELL NO. 2 (Well K-9), finished in sand and gravel, was completed in June 1945 to a depth of 95 ft (reported in March 1960 at 92 ft deep) by Henry Boysen, Jr., Libertyville. The well is located at the corner of Eastway and Forest Drives, approximately 1385 ft S and 1255 ft E of the NW corner of Section 21, T44N, R9E, Lake County. The land surface elevation at the well is approximately 770 ft.

A drillers log of Well No. 2 follows:

Strata	Thickness (ft)	Depth (ft)
Gravel	36	36
Sand	48	84
Gravel	11	95

An 8-in. diameter hole was drilled to a depth of 95 ft. The well is cased with 8-in. steel pipe from 1 ft above land surface to a depth of 84 ft followed by 11 ft (10 ft slotted) of 8-in. No. 14 slot Cook red brass screen.

Upon completion, after pumping for 2 days, the well reportedly produced 280 gpm with a drawdown of 16 ft from a nonpumping water level of 9 ft below land surface.

This well was acidized in March 1960 by the J. P. Miller Artesian Well Co., Brookfield, and the yield was reportedly improved from 75 to 450 gpm. A production test was conducted by the J. P. Miller Artesian Well Co. on March 7, 1960. After 4 hr of pumping at rates of 110 to 360 gpm, the final drawdown was 21 ft from a nonpumping water level of 19 ft below land surface.

On May 20, 1963, the nonpumping water level was reported to be 10 ft.

On July 24, 1959, after 20 min of pumping at a rate of 300 gpm, the drawdown was 75 ft from a nonpumping water level of 227 ft.

The pumping equipment presently installed consists of a 60-hp 1760 rpm Louis Allis electric motor (Type OGX, No. 2381057), a 10-in., 12-stage Byron Jackson turbine pump (No. 344228) set at 340 ft, rated at 300 gpm at about

480 ft TDH, and has 340 ft of 5-in. column pipe. The well is equipped with 340 ft of airline.

A mineral analysis of a sample (Lab. No. 150133) collected July 23, 1959, after pumping for 20 min at 300 gpm, showed the water to have a hardness of 244 mg/l, total dissolved minerals of 335 mg/l, and an iron content of 0.2 mg/l. Hydrogen sulfide gas was apparent when this sample was collected.

LAKE BARRINGTON SHORES SUBDIVISION

Lake Barrington Shores Subdivision (est. 135), located 0.2 mile north of North Barrington, installed a public water supply in 1974. The water system is owned and operated by the Lake Barrington Community Homeowners Association. One well is in use. In 1975 there were 72 services, none metered; the average and maximum daily pumpages were 30,200 and 38,600 gpd, respectively. The water is chlorinated and treated with polyphosphate to keep iron in solution.

WELL NO. 1, finished in sand and gravel, was completed in March 1973 to an effective depth of 127 ft by the Layne-Western Co., Aurora. The well is located in the southeast portion of the subdivision, approximately 500 ft N and 600 ft W of the SE corner of Section 11, T43N, R9E. The land surface elevation at the well is approximately 815 ft.

A drillers log of Well No. 1 follows:

Strata	Thickness (ft)	Depth (ft)
Brown sandy top soil	1	1
Brown sandy clay	1.5	2.5
Brown clayey sand and gravel	1.5	4
Brown sandy clay with gravel embedded, boulders	21	25
Gray clay, firm with some gravel and boulders	42	67
Gray clay, firm	5	72
Lime ledges, boulder, clay	4	76
Coarse sand and gravel, boulders	14	90
Coarse sand — medium gravel, boulders	8	98
Fine — medium sand and gravel, tight	21	119
Fine — medium sand and gravel, loose	10	129

A 38-in. diameter hole was drilled to a depth of 129 ft. The well is cased with 16-in. pipe from land surface to a depth of 86 ft and from 96 ft to 117 ft and the screened sections consist of 16-in. diameter No. 5 (0.105 in.) Layne stainless steel shutter from 86 ft to 96 ft and from 117 ft to 127 ft. The annulus between the bore hole and casing-screen

assembly is filled with cement from 0 to 20 ft, with clay from 20 to 65 ft, and with No. 3 Muscatine gravel from 65 to 129 ft.

A production test was conducted by the driller on March 6-7, 1973. After 24 hr of pumping at rates of 510 to 596 gpm, the drawdown was 18 ft from a nonpumping water level of 55 ft below land surface.

The pumping equipment presently installed consists of a 50-hp 1765 rpm U.S. electric motor, a 10-in., 4-stage Layne turbine pump set at 100 ft, rated at 600 gpm at about 196 ft TDH, and has 100 ft of 8-in. column pipe.

The following mineral analysis made by the Illinois Environmental Protection Agency (Lab. No. C005514) is for a water sample from the well collected January 22, 1975, after 30 min of pumping at 600 gpm.

WELL NO. 1, LABORATORY NO. C005514

		mg/l	me/l			mg/l	me/l
Iron	Fe	1.3		Silica	SiO$_2$	26.0	
Manganese	Mn	0.04		Fluoride	F	0.6	0.03
Ammonium	NH$_4$	0.87	0.05	Boron	B	0.5	
Sodium	Na	47	2.04	Nitrate	NO$_3$	1.4	0.02
Potassium	K	1.8	0.05	Chloride	Cl	3	0.08
Calcium	Ca	84	4.19	Sulfate	SO$_4$	275	5.72
Magnesium	Mg	62	5.10	Alkalinity (as CaCO$_3$)		302	6.04
Arsenic	As	0.000					
Barium	Ba	0.2		Hardness (as CaCO$_3$)		467	9.34
Copper	Cu	0.00					
Cadmium	Cd	0.00		Total dissolved			
Chromium	Cr	0.00		minerals		722	
Lead	Pb	0.02					
Mercury	Hg	0.0000		pH (as rec'd)		8.1	
Nickel	Ni	0.0		Radioactivity			
Selenium	Se	0.00		Alpha pc/l		0.7	
Silver	Ag	0.00		±deviation		1.7	
Cyanide	CN	0.00		Beta pc/l		3.5	
Zinc	Zn	0.02		±deviation		2.4	

LAKE BLUFF

The village of Lake Bluff (4979) installed a public water supply in 1904. A total of three production wells were utilized as a source of water supply until 1960 when the village began purchasing Lake Michigan water from the city

of Lake Forest. One well (No. 3) is available for emergency use. In 1956 there were 730 services, all metered; the average and maximum daily groundwater pumpages were 150,000 and 180,000 gpd, respectively. In 1972 there were

1388 services, all metered; the average daily surface water pumpage was 600,000 gpd.

Water was first obtained from a well variously reported at 1600, 1900, and 2000 ft in depth and 4 in. in diameter. It was purchased by the village in 1896 from the Lake Bluff Camp Meeting Association and was probably drilled between 1880 and 1890. This well furnished the entire public water supply until 1908 and a considerable part of the supply until 1913. It was reported that the original static water level was 45 ft above land surface, and in 1919 was 45 ft below land surface and the yield 75 gpm. The well was sounded in 1921 and found to have a depth of 450 ft. In 1924 it was again sounded and found to have a depth of 250 ft. The well was then completely filled and abandoned.

A second well, 50 ft from the old well and 300 ft in depth, was drilled in 1908. It was reported to be 6 in. in diameter and the bottom 100 ft penetrated rock. The water from this well was reported to be of better quality than water from the deeper sandstone well, but its yield capability was limited. It was abandoned about 1914.

WELL NO. 1 (originally No. 3), finished in Silurian dolomite, was completed in 1913 to a depth of 498 ft by William Cater, Chicago. This well was abandoned and sealed in 1964. The well was located at Center Ave. and Park Drive, approximately 1700 ft S and 500 ft E of the NW corner of Section 21, T44N, R12E. The land surface elevation at the well is approximately 680 ft.

A 10-in. diameter hole was drilled to a depth of 350 ft and finished 8 in. in diameter from 350 to 498 ft. The well was cased with 10-in. pipe from 0.8 ft above the concrete floor in the pump foundation base to a depth of 197 ft.

In June 1924, the nonpumping water level was reported to be 55 ft below the pump base.

On January 1, 1933, a production test was conducted for 3 hr during which time the average rates of pumpage were 94.5 gpm for the first hr, 86.8 gpm for the second hr, and 70.2 gpm the third hr. The nonpumping water level was reported to be 87 ft below the pump base.

A mineral analysis of a sample (Lab. No. 63774) collected April 3, 1929, showed the water to have a hardness of 91 mg/l, total dissolved minerals of 295 mg/l, and an iron content of 0 mg/l.

WELL NO. 2 (originally No. 4), finished in the Mt. Simon Sandstone, was completed in 1921 to a depth of 1804 ft by William Cater, Chicago. This well was capped and disconnected in 1964. The well is located at Center Ave. and Park Drive, approximately 1700 ft S and 460 ft E of the NW corner of Section 21, T44N, R12E. The land surface elevation at the well is approximately 680 ft.

A 12-in. diameter hole was drilled to a depth of 670 ft, reduced to 8 in. between 670 and 1256 ft, and finished 6.2 in. in diameter from 1256 to 1804 ft. The well was cased from 1.5 ft above the pumphouse floor to an unknown depth.

Upon completion, the well produced 134 gpm for 1 hr

with a drawdown of 18 ft from a nonpumping water level of 58 ft below the top of the casing.

A production test was conducted on March 21, 1950, by representatives of the village and the State Water Survey. After 3 hr of pumping at rates of 270 to 362 gpm, the final drawdown was 28.6 ft from a nonpumping water level of 148.5 ft below the pump base. Thirty min after pumping was stopped, the water level had recovered to 150.3 ft.

Nonpumping water levels reported for this well are: 156 ft below land surface on November 28, 1951; 210 ft on October 22, 1958; and 245.1 ft below land surface on November 16, 1964.

Monthly measurements of the nonpumping water levels during the period January 1965 to January 1975 show a regional decline from about 242 to 332 ft below land surface.

A correlated drillers log of Well No. 2 furnished by the State Geological Survey follows:

Strata	Thickness (ft)	Depth (ft)
PLEISTOCENE SYSTEM		
Clay	55	55
Quicksand	7	62
Gravel	5	67
Clay	45	112
Gravel	4	116
Clay	65	181
Gravel	3	184
SILURIAN SYSTEM		
Niagaran–Alexandrian Series		
Dolomite	308	492
ORDOVICIAN SYSTEM		
Maquoketa Group		
Shale, some dolomite	138	630
Galena–Platteville Group		
Rock	304	934
Glenwood–St. Peter Formations		
Sand rock	212	1146
CAMBRIAN SYSTEM		
Franconia and Galesville Formations		
Shale, blue	54	1200
Sand	204	1404
Eau Claire and Mt. Simon Formations		
Pencil rock		1404
Lime, shale and sand	261	1665
Sand	139	1804

A mineral analysis of a sample (Lab. No. 107458) collected August 21, 1946, after pumping for 40 min at 500 gpm, showed the water to have a hardness of 374 mg/l, total dissolved minerals of 512 mg/l, and an iron content of 0.5 mg/l.

WELL NO. 3, open to the Galena–Platteville Dolomite and the Glenwood–St. Peter, Ironton–Galesville, and Mt. Simon Sandstones, was completed in November 1956 to a depth of 1828 ft by L. Cliff Neely, Batavia. This well is available for emergency use. The well is located about 760 ft west of Well No. 2, approximately 1700 ft S and 300 ft W of the NE corner of Section 20, T44N, R12E. The land surface elevation at the well is approximately 685 ft.

A 30-in. diameter hole was drilled to a depth of 180 ft, reduced to 25 in. between 180 and 665 ft, reduced to 19 in. between 665 and 1260 ft, reduced to 15 in. between 1260 and 1693 ft, and finished 12 in. in diameter from 1693 to

43

	Thickness	Depth
Strata (continued)	(ft)	(ft)
Gravel, clay and sand	25	236
Clay and some gravel	13	249
Gravel	9	258
SILURIAN SYSTEM		
Dolomite, blue, white, brown	10	268
Dolomite, gray	4	272
Dolomite, brownish gray	41	313

A 10-in. diameter hole was drilled to a depth of 313 ft.
The well is cased with 10-in. black steel pipe to a depth of
260 ft. The top of the well casing is equipped with a pitless
adapter.

Upon completion, the well reportedly produced 100 gpm
for 8 hr with a drawdown of 74 ft from a nonpumping water
level of 46 ft below land surface.

On January 12, 1972, the nonpumping water level was re-
ported to be 54 ft.

The pumping equipment presently installed is a Goulds sub-
mersible pump (Model No. UD66L-X32) set at 240 ft, and
powered by a 10-hp Franklin Electric motor.

A mineral analysis made by the Illinois Environmental
Protection Agency (Lab. No. C004852) of a sample collected
January 8, 1974, after pumping for 1.2 hr at 160 gpm,
showed the water to have a hardness of 204 mg/l, total dis-
solved minerals of 458 mg/l, and an iron content of 0.1 mg/l.

WELL NO. 3, finished in Silurian dolomite, was com-
pleted in August 1958 to a depth of 330 ft by the Hoover
Water Well Service, Zion. The well is located south of the
Milwaukee RR tracks on Clifton Ave., approximately 1800
ft N and 600 ft E of the SW corner of Section 28, T45N,
R10E. The land surface elevation at the well is approxi-
mately 795 ft.

A 12-in. diameter hole was drilled to a depth of 330 ft.
The well is cased with 12-in. pipe from 1.5 ft above the
pumphouse floor to a depth of 237 ft.

A production test was conducted by the driller on August
29, 1958. After 4 hr of pumping at a rate of 350 gpm, the
drawdown was 10 ft from a nonpumping water level of 78
ft below land surface. Pumping was continued for an addi-
tional 8 hr at 500 gpm with a final drawdown of 20 ft.

On January 12, 1972, the well reportedly produced 470
gpm with a drawdown of 8 ft from a nonpumping water
level of 81 ft.

The pumping equipment presently installed consists of a
40-hp 1800 rpm U.S. electric motor (Serial No. 153698), a
10-in., 7-stage Johnston oil-lubricated turbine pump (Serial
No. JN-4252) set at 200 ft, rated at 350 gpm at about 375 ft
head, and has 200 ft of 6-in. column pipe. A 10-ft section
of 6-in. suction pipe is attached to the pump intake. The
well is equipped with 200 ft of airline.

The following mineral analysis made by the Illinois Envi-
ronmental Protection Agency (Lab. No. 02422) is for a water
sample from the well collected October 26, 1971, after 30
min of pumping at 470 gpm.

WELL NO. 3, LABORATORY NO. 02422

		mg/l	me/l			mg/l	me/l
Iron	Fe	0.0		Silica	SiO$_2$	14	
Manganese	Mn	0.0		Fluoride	F	1.02	0.05
Ammonium	NH$_4$	0.46	0.02	Boron	B	0.6	
Sodium	Na	66.0	2.87	Nitrate	NO$_3$	0	
Potassium	K	0.7	0.02	Chloride	Cl	4.0	0.11
Calcium	Ca	32.8	1.64	Sulfate	SO$_4$	163	3.38
Magnesium	Mg	21.0	1.73	Alkalinity (as CaCO$_3$)		128	2.56
Barium	Ba	0.0		Hardness (as CaCO$_3$)		160	
Copper	Cu	0.0		Total dissolved			
Cadmium	Cd	0.00		minerals		380	
Chromium	Cr	0.0		pH (as rec'd)	8.0		
Lead	Pb	0.00		Radioactivity			
Mercury	Hg	<0.0005		Alpha pc/l	0		
Nickel	Ni	0.05		±deviation	0		
Silver	Ag	0.0		Beta pc/l	0		
Zinc	Zn	0.0		±deviation	1		

STRAWBERRY 1 CONDOMINIUM DEVELOPMENT

Strawberry 1 Condominium Development (est. 126), lo-
cated 1 mile southwest of North Chicago, installed a public
water supply in 1973. The water system is owned and op-
erated by the Strawberry 1 North Chicago Association. Two
wells are in use. In 1974 there were 90 services, all metered;
the average and maximum daily pumpages were 7600 and
11,400 gpd, respectively. The water is chlorinated and
fluoridated.

WELL NO. 1, finished in Silurian dolomite, was com-
pleted in September 1972 to a depth of 215 ft by the Henry
Boysen Co., Libertyville. The well is located about 25 ft from
the pump control and treatment building, approximately
2000 ft S and 1600 ft E of the NW corner of Section 7, T44N,
R12E. The land surface elevation at the well is approximately
685 ft.

A 6-in. diameter hole was drilled to a depth of 215 ft.
The well is cased with 6-in. galvanized pipe from land sur-
face to a depth of 215 ft. The top of the well casing is
equipped with a pitless adapter.

Upon completion, the well reportedly produced 20 gpm
with very little drawdown from a nonpumping water level of
99 ft below land surface.

The pumping equipment presently installed is a Red
Jacket submersible pump set at 168 ft, rated at 50 gpm,
and powered by a 5-hp Red Jacket electric motor.

A drillers log of Well No. 1 follows:

	Thickness	Depth
Strata	(ft)	(ft)
Backfill and red clay	10	10
Sand	4	14
Gray sandy clay	9	23

74

Strata (continued)	Thickness (ft)	Depth (ft)
Sand	5	28
Gray clay	45	73
Sand	6	79
Gray clay	11	90
Sand	10	100
Hardpan clay	73	173
Limestone	42	215

The following mineral analysis (Lab. No. 195672) is for a water sample from the well collected May 21, 1974.

WELL NO. 1, LABORATORY NO. 195672

		mg/l	me/l			mg/l	me/l
Iron (total)	Fe	1.5		Silica	SiO$_2$	10.0	
Manganese	Mn	0.01		Fluoride	F	0.6	
Ammonium	NH$_4$	0.2	0.01	Boron	B	0.7	
Sodium	Na	103	4.48	Nitrate	NO$_3$	0.3	Tr
Potassium	K	1.5	0.04	Chloride	Cl	14	0.39
Calcium	Ca	36.0	1.80	Sulfate	SO$_4$	255.5	5.31
Magnesium	Mg	18.5	1.52	Alkalinity (as CaCO$_3$)		102	2.04
Strontium	Sr	1.36	0.03				
Barium	Ba	<0.1		Hardness (as CaCO$_3$)		166	3.32
Copper	Cu	0.00					
Cadmium	Cd	0.00		Total dissolved			
Chromium	Cr	0.00		minerals		498	
Lead	Pb	<0.05		Turbidity	13		
Lithium	Li	0.01		Color	0		
Nickel	Ni	<0.05		Odor	0		
Zinc	Zn	0.17		Temp.	51.5 F (reported)		

WELL NO. 2, finished in Silurian dolomite, was com-

pleted in October 1972 to a depth of 295 ft by the Henry Boysen Co., Libertyville. The well is located about 500 ft southwest of Well No. 1, approximately 2500 ft S and 1400 ft E of the NW corner of Section 7, T44N, R12E. The land surface elevation at the well is approximately 685 ft.

A drillers log of Well No. 2 follows:

Strata	Thickness (ft)	Depth (ft)
Backfill and clay, red	10	10
Sand	4	14
Gray clay	17	31
Sand	8	39
Gray hard clay, sandy	46	85
Sand	10	95
Hardpan clay	81	176
Limestone	94	270
Shale	25	295

A 6-in. diameter hole was drilled to a depth of 295 ft. The well is cased with 6-in. galvanized pipe from land surface to a depth of 295 ft. The top of the well casing is equipped with a pitless adapter.

Upon completion, the well reportedly produced 20 gpm with very little drawdown from a nonpumping water level of 99 ft below land surface.

The pumping equipment presently installed is a Red Jacket submersible pump set at 189 ft, rated at 50 gpm, and powered by a 5-hp Red Jacket electric motor.

STURM SUBDIVISION

Sturm Subdivision (est. 63), located about 2 miles southeast of Lake Zurich, installed a public water supply in 1957. One well is in use. In 1973 there were 18 services, none metered; the estimated average and maximum daily pumpages were 3780 and 5700 gpm, respectively. The water is not treated.

WELL NO. 1, finished in Silurian dolomite, was completed in October 1957 to a depth of 295 ft by the Hoover Water Well Service, Zion. The well is located at the end of Sturm St. in a turnaround circle, approximately 300 ft S and 2590 ft W of the NE corner of Section 33, T43N, R10E. The land surface elevation at the well is approximately 830 ft.

A drillers log of Well No. 1 follows:

Strata	Thickness (ft)	Depth (ft)
Top soil	1	1
Yellow clay	14	15
Blue clay	115	130
Blue hardpan	50	180
Sandy blue clay, some gravel	40	220
Mixed clay and gravel	17	237
Soft Niagra lime	1	238
Light brownish lime	57	295

A 10-in. diameter hole was drilled to a depth of 238 ft and finished 8 in. in diameter from 238 to 295 ft. The

well is cased with 8-in. pipe from 1.7 ft above the pumphouse floor to a depth of 238 ft.

A production test was conducted by the driller on October 7, 1957. After 12 hr of pumping at rates of 301 to 496 gpm, the final drawdown was 71 ft from a nonpumping water level of 89 ft.

The pumping equipment presently installed consists of a 1-hp Franklin electric motor, a 4-in., Fairbanks–Morse submersible pump (Model No. 100S22) set at 126 ft, rated at 15 gpm, and has 126 ft of 1-in. column pipe.

The following mineral analysis (Lab. No. 195670) is for a water sample from the well collected May 14, 1974.

WELL NO. 1, LABORATORY NO. 195670

		mg/l	me/l			mg/l	me/l
Iron (total)	Fe	0.3		Silica	SiO$_2$	18.3	
Manganese	Mn	0.01		Fluoride	F	0.7	
Ammonium	NH$_4$	0.0	0.00	Boron	B	0.5	
Sodium	Na	71.2	3.10	Nitrate	NO$_3$	2.9	0.05
Potassium	K	2.3	0.06	Chloride	Cl	1	0.03
Calcium	Ca	86.4	4.31	Sulfate	SO$_4$	423.9	8.82
Magnesium	Mg	52.2	4.29	Alkalinity (as CaCO$_3$)		136	2.72
Strontium	Sr	2.09	0.05				
Barium	Ba	<0.1		Hardness (as CaCO$_3$)		430	8.60
Copper	Cu	0.02					
Cadmium	Cd	0.00		Total dissolved			
Chromium	Cr	0.00		minerals		772	
Lead	Pb	<0.05		Turbidity	1		
Lithium	Li	0.01		Color	0		
Nickel	Ni	<0.05		Odor	0		
Zinc	Zn	0.03		Temp.	53.5 F (reported)		

APPENDIX C

TABLE C-1

PHYSICAL LABORATORY TEST RESULTS[1]
SLURRY WALL BACKFILL MATERIAL
FAIRCHILD SAN JOSE FACILITY

Sample Number	Date Sampled	Station Number	Permeability cm/sec
9	10/28/85	11+50 to 18+13	7.3×10^{-9}
10	10/29/85	N/A	9.8×10^{-9}
29	12/31/85	13+75	8.2×10^{-9}
N/A	N/A	18+00	9.0×10^{-8}
N/A	N/A	19+75	1.3×10^{-7}
43	02/20/86	20+75	1.7×10^{-9}
N/A	N/A	26+50	9.2×10^{-8}
59	03/18/86	33+50	1.0×10^{-8}
61	04/05/86	1+50	1.0×10^{-8}

Average Permeability 4.0×10^{-8}

(1) Report: Soil-Bentonite Cutoff Wall, San Jose Facility, November, 1986

CanonieEnvironmental

SAN JOSE SLURRY WALL

The slurry wall ranges in depth from 75 to 130 feet, has a total perimeter of 3,454 feet, and had 160,000 square feet of wetted area during the pumping tests.[1] Figures N-16[1] and N-17[1] graphically relate the pumping rate from inside the San Jose slurry wall enclosure to the ground water elevation inside the slurry wall enclosure.

The slurry wall creates an effective barrier for the movement of ground water by reducing the ground water velocity through the wall. Although the ground water velocity is proportional to permeability, it is also inversely proportional to the thickness of the wall and directly proportional to the differential head across the wall. Permeability is not the sole factor that determines the effectiveness of a slurry wall.

[1]Draft Report, Remedial Action Plan, Fairchild Semiconductor Corporation, San Jose Facility, Volume 5 of 5, Appendix N.

CanonieEnvironmental

TIME vs. PUMPING RATE at WELL WCC-20(B)

82-012-B517

Pumping resumed
March 10, 1987
with larger capcity pump

Pumping ended
March 2, 1987

FIGURE N-16
CanonieEnvironmental

$\frac{dh}{dt}$ SLOPE ANALYSIS OF
GROUND WATER ELEVATIONS vs. TIME at WELL WCC-5(B)

FIGURE N-I7

CanonieEnvironmental

% BENTONITE BY DRY WEIGHT OF SB BACKFILL

(After D'Appolonia 1980)

JOB NO. 863-3389	SCALE AS SHOWN	PERMEABILITY OF SOIL BENTONITE SLURRY	
DRAWN LJW	DATE 9/8/87		
CHECKED *Bw*	DWG NO. 12		
Golder Associates		OUTBOARD MARINE CORP.	FIGURE 1